DESIGN OF TAINTER GATE PRESTRESSED CONCRETE TRUNNION ANCHORAGES

By

William Preston Johnson, Jr.

Bachelor of Arts
The Rice University
Houston, Texas
1957

Bachelor of Science
The Rice University
Houston, Texas
1958

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DESIGN OF TAINTER GATE

PRESTRESSED CONCRETE

TRUNNION ANCHORAGES

Thesis Approved:

Roger L. Flanders

Thesis Adviser

J. W. Gillespie

J. H. Boggs

Dean of the Graduate School

PREFACE

It is the author's wish that this thesis will bring to light some of the problems in tainter gate and trunnion anchorage design. This is the first time most of the enclosed material has been assembled and should be of value to future designers of such projects.

The author wishes to acknowledge and express his indebtedness and gratitude to the following individuals:

To Wilton O. Dixon, Chief, Structural Section "R", Tulsa District Office, U. S. Corps of Engineers, and Larry R. Brown, Assistant Chief of the section for their suggestions, encouragement, and guidance during the development of the Millwood project.

To Homer G. Russell and Frank E. Webster, structural engineers who also worked on the Millwood Dam design.

To the Tulsa District Office, U. S. Corps of Engineers, who have given permission to use the material developed while in their employ.

To Professor Roger L. Flanders for being my major advisor on this Thesis.

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NOMENCLATURE

A_{bng} = Bearing area in square inches between the thrust girder and the downstream face of the pier.

A_t = Cross sectional area of the thrust girder.

e = Eccentricity of the transverse prestress force.

e' = Eccentricity of the horizontal trunnion reactions.

F = Final prestress force per prestress bar.

F_L = Final total longitudinal prestress force.

F_t = Final total transverse prestress force.

f_{bng} = Average bearing stress.

f_t = Average transverse prestress.

M = Maximum moment in thrust girder.

N_t = Number of required transverse prestress bars.

R_H = Horizontal trunnion reaction.

S_n = Principal tensile stress.

S_{pier} = Section modulus of the pier bearing area.

S_{TG} = Section modulus of thrust girder.

v_{avg} = Average shear stress across the thrust girder.

CHAPTER I

INTRODUCTION

1-1 Tainter Gates

The general purpose of a flood control dam is to store the excess water of a given period and release it at a more favorable time. These dams can take many forms, but one of the more common types is the controlled spillway. This is a dam where the water flow over the spillway is controllable. One means of controlling the flow was invented by Captain Jeremiah B. Tainter of Menominee, Wisconsin. This widely used device bearing the inventor's name was patented in 1886 (1).

The following is a definition of a tainter gate:

> A crest gate whose face is a section of a cylinder, which rotates about a horizontal axis downstream from the gate, the water pressure against the gate being concentrated in the axis, reducing friction in raising and lowering the gate; a seal is placed along the sides and bottom of the gate face for water tightness. The gate is raised and lowered by winches or hoists attached to cables or chains fastened to the bottom edge of the gate and lying against its water face, which allows a vertical lifting force to be applied. It can be closed under its own weight (2).

1-2 Utilization of Tainter Gates

Since the invention of tainter gates, they have been used in a variety of applications and on numerous structures. The only designs discussed in this paper will be crest gates on overflow weirs. The Tulsa District Office, Corps of Engineers, has designed and/or constructed eleven dams utilizing one hundred forty-six gates. Two other dams with fifty-nine gates have been built by the Grand River Dam Authority in the same area. Now being designed for construction in the immediate future are five dams with fifty gates. A list of these projects and gate sizes is given in Table I.

Some other major installations of tainter gates are in the Columbia River Basin. A partial list of these dams and gates is given in Table II (3).

The number and size of these gates proves their popularity.

Some of the reasons tainter gates are largely used are:

1. Relative ease of steel design
2. Ease of fabrication
3. All welded construction
4. Sealing not difficult
5. Low hoist loads
6. Ease of installation
7. Economy.

1-3 Nomenclature of a Tainter Gate

A tainter gate is generally composed of the following

TABLE I

TAINTER GATES ON DAMS IN THE TULSA DISTRICT

Name of Dam	No. of Gates	Width	Height	Radius	Pier Width
Hulah	10	40'-0"	26.10'	26'-0"	8'-0"
Canton	16	40 -0	25.24	26 -0	8 -0
Fall River	8	50 -0	26.55	25 -6	10 -0
Pensacola (Main)	21	36 -0	25.	24 -0	5 -0
Pensacola (East)	21	37 -0	15.	15 -0	4 -0
Ft. Gibson	30	40 -0	35.46	35 -0	10 -0
Markham Ferry	17	40 -0	37.	36 -0	7 -9
Tenkiller	10	50 -0	25.89	25 -6	10 -0
Toronto	8	40 -0	25.52	30 -0	8 -0
Keystone*	18	40 -0	36.19	36 -6	8 -0
Eufaula*	11	40 -0	32.73	33 -0	8 -0
John Redmond*	14	40 -0	36.19	36 -0	8 -0
Millwood**	13	40 -0	32.39	33 -0	8 -0
Broken Bow**	8	40 -0	40.12	40 -0	8 -0
Being Designed					
Pine Creek**	7	50 -0	45.5	45 -6	10 -0
Gillham**	10	40 -0	40	40 -0	8 -0
Marion**	3	40 -0	40	40 -0	8 -0
Robert S. Kerr**	18	50 -0	44	44 -0	10 -0
Webers Falls**	12	50 -0	41	41 -0	10 -0

*Dams utilizing prestressed trunnion anchorages.

**Dams utilizing prestressed concrete trunnion anchorages.

TABLE II

SPILLWAY TAINTER GATES OF THE COLUMBIA RIVER BASIN

Name of Dam	No. of Gates	Width	Height
Big Cliff	3	46	46
Brownlee*	4	32	50
Chief Joseph	19	40	46
Cougar	2	40	43
Detroit	6	42	35
Dexter	7	42	38
Eagle Gorge	2	45	30
Hills Creek	3	42	48
Ice Harbor	10	50	53
John Day*	20	50	59.95
Lookout Point	5	42.5	41.5
Lower Monumental**	8	50	60.56
Oxbow*	6	32	50
Priest Rapids	22	40	50
Rocky Reach	12	50	58
Swift	2	39	30
The Dalles	23	50	42.5
Wanapum**	12	50	65
Yale	5	39	30

*Dams utilizing prestressed trunnion anchorages.

**Dams utilizing prestressed concrete trunnion anchorages.

Source: (3)

parts. Variations in designs may change some components.

Skinplate. The skinplate generally varies from three-eighth inch thick at the top of the gate to possibly five-eighth inch thick plate at the bottom of the gate.

Vertical Ribs. The vertical ribs are usually a structural tee section rolled to the radius of the gate. The leg of the tee is welded to the downstream side of the skinplate. A variation of design may have horizontal ribs with two vertical girders.

Horizontal Girders. There are generally two to four horizontal girders of either built up or rolled sections. The upstream flanges are welded to the flanges of the ribs. Since these girders have their webs nearly horizontal, the downstream flanges need a system of supports.

End Frames. The end frames may be either parallel or inclined to the pier face. The end frames consist of the legs or struts from each girder and some connecting bracing. The inclined end frames result in a more economical design.

Trunnion Assembly. This heavy assembly is at the concurrence of the struts. The struts are welded to a cylindrical steel forging called the trunnion barrel.

Trunnion Pin and Yoke. The trunnion pin fits inside the trunnion barrel and along with the yoke forms the "hinge" for the gate. The yoke is attached to the anchorage and must be accurately located and firmly held in place to permit smooth operation of the gate.

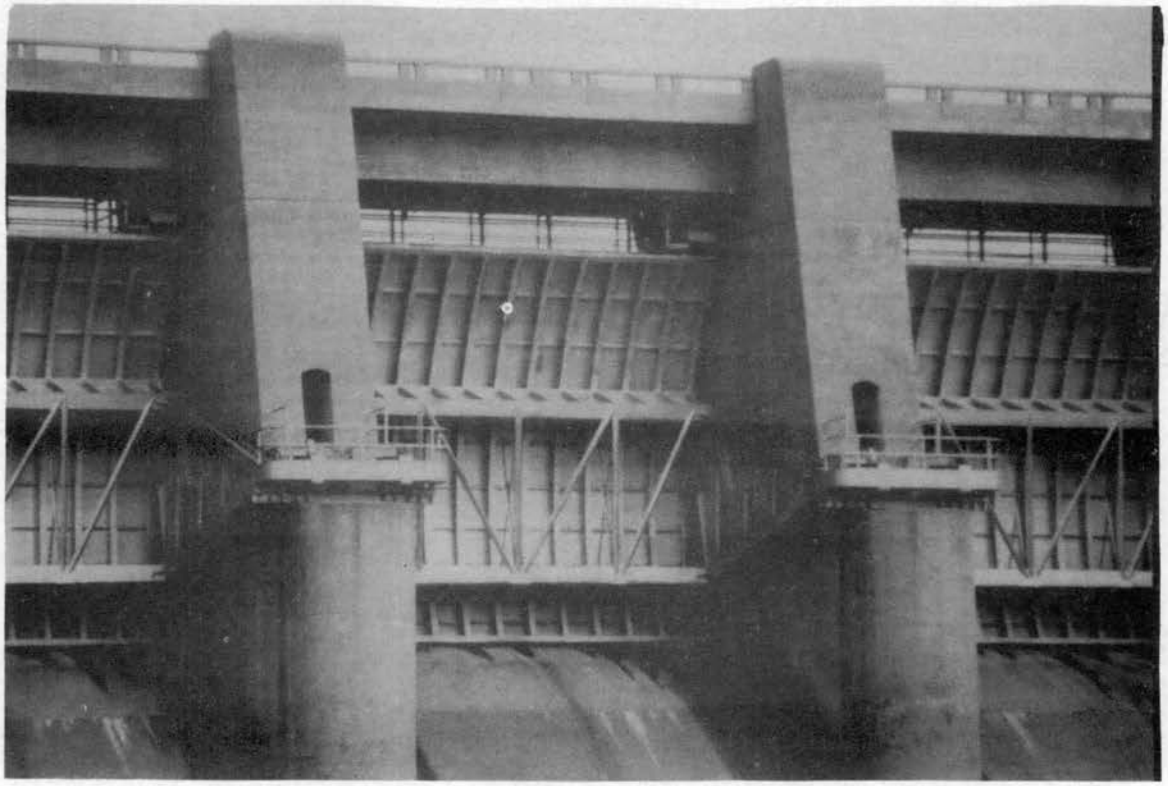


Figure 1-1a. Fort Gibson Dam Tainter Gates

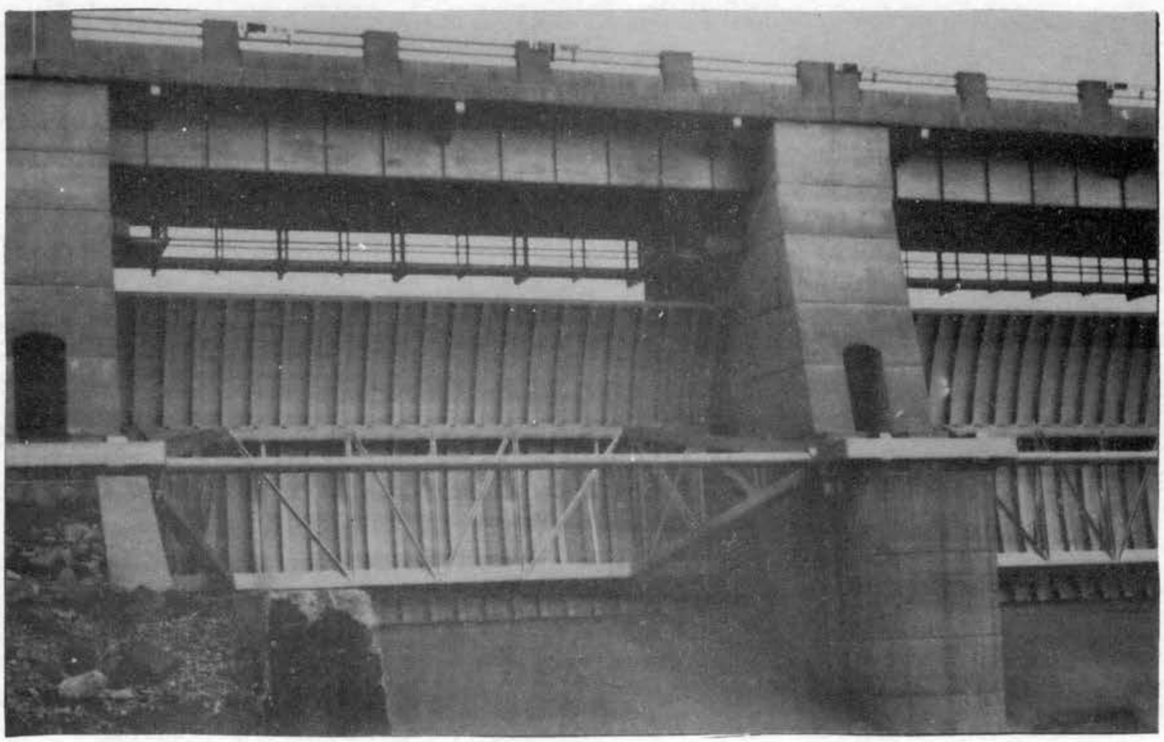


Figure 1-1b. Tenkiller Dam Tainter Gates

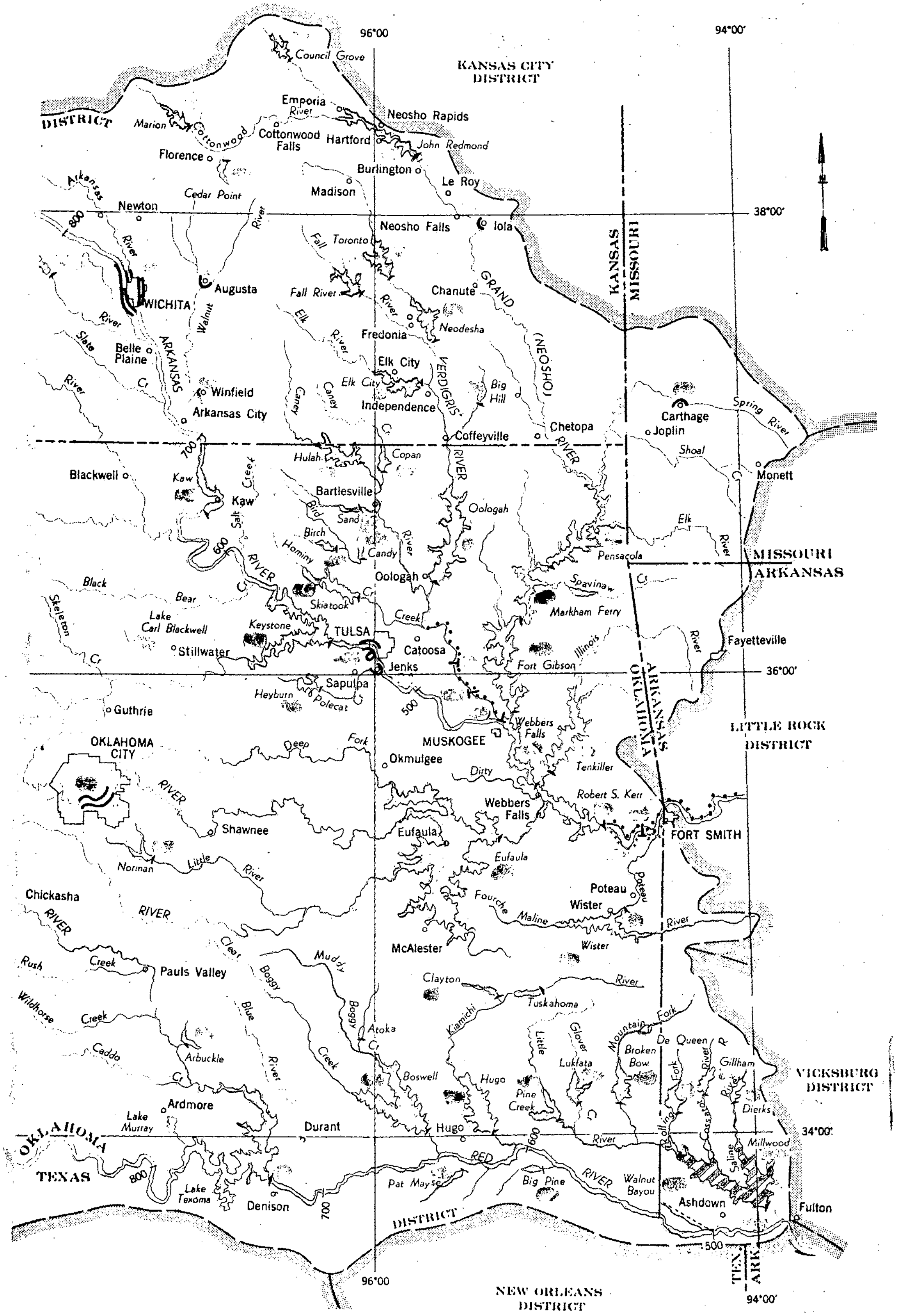


Figure 1-2. Map - Tulsa District

Anchorages. The anchorage must take the concentrated loads from the trunnion yoke and transfer them to the rest of the dam. The design of these anchorages has changed in the last few years with the availability of high strength prestressing bars.

1-4 Statement of Problem

Although many dams and gates have been built very little has been written about their design. Most of the designers have relied on either the Corps of Engineer Manual on Tainter Gates, a manual by the Bureau of Reclamation, or their own experience (4). In this thesis, the author assembled information that has not been available to designers. The development of the prestressed concrete trunnion anchorage and the accompanying data will be of great value to future designers.

This thesis gives a brief summary of some of the problems in the anchorage design. The recent use of prestressing bars in several installations is illustrated. An anchorage design similar to the author's development of the Millwood Dam prestressed concrete trunnion anchorage is given as sample calculations. These calculations are applicable to the design of any anchorage of this type. This anchorage is presently the standard of the Corps of Engineers. From the summary of actual bids on the different types of anchorages, it is shown that the prestressed concrete trunnion anchorage of the Millwood type is the least costly that has been designed and built by the Tulsa District.

CHAPTER II

ANCHORAGES

2-1 Loads on Tainter Gate Anchorages

The anchorage system of a tainter gate must be capable of accepting a variety of loads. These loads come directly through the trunnion yoke. The magnitude and direction of these loads varies greatly. Principally there is a large horizontal load component. Depending on the position of the trunnion pin in relation to the rest of the gate and the height of water on the gate, there will be a vertical load component. This vertical component can be either up or down. It is generally of a small magnitude. The vertical load can become appreciable if the trunnion pin cannot be located near its ideal position or if the gate is operated in the surcharged condition. A surcharged gate is a gate partially open with the water level remaining at the top of the gate.

If the end frames of the gate are sloping, then a large thrust is induced normal to the face of the pier. This thrust can have the magnitude of one-fourth the value of the loads parallel to the pier face. If the end frames are parallel to the pier face, the thrust into the pier is considerably reduced but a binding moment is introduced in the trunnion pin. At the intermediate piers, all of these loads are usually balanced by the simultaneous application of adjacent gates. However, the possibility remains as a design condition, of having one gate inoperable and therefore closed and fully loaded while the adjacent gate is entirely open and has a very small horizontal trunnion reaction. This condition is similar to that of each end pier of the dam.

Before the anchorage can be designed, the trunnion loads have to be calculated. For any gate, the horizontal and vertical reactions at the trunnion can be calculated from statics. A hydrostatic force polygon can be drawn for any gate opening and any water level. The amount of thrust into the pier can only be approximated from the properties of the gate.

In the case of a three horizontal girder gate, the design proceeds like this. The skinplate is designed as a continuous member over each rib. The ribs are assumed to be straight members continuous over three unyielding supports and loaded with the developed hydrostatic load. This load may be approximated by a triangular loading. Another method is to find the actual water pressure at each girder or support. This forms a series of trapizoidal loading diagrams that more closely approach the true loading diagram.

The reaction at each support is taken as the load on the corresponding girder. This load is slightly greater than the actual load on the girder due to the curvature of the ribs. Each girder and corresponding struts or legs can now be designed as a rigid frame. When all three frames have been solved, the thrusts into the pier can be summed. The horizontal and vertical trunnion reactions can also be vectorially summed from the three frames. These total vertical and horizontal reactions will be slightly higher than those obtained from the statical solution of the gate loads.

Other loads that can be imposed on the gate are the sill reaction and a wave action. The sill reaction is a force exerted on the bottom edge of the gate when the gate is resting on the weir of the dam. If the spillway is so located that there will be a wave action against the gates, this load must be added to the normal hydrostatic load. For this condition, a stress of one-third greater than the normal working stress is allowed. In the most economical rib design, the horizontal girders are spaced so that the design negative moments at each girder are equal. For a gate with a large wave loading, the girders can be so spaced that the top cantilever moment will be less than one and one-third times the negative moment at any girder under normal loading conditions. However, if an effective length of the cantilevered rib is assumed to be longer than the actual length, this moment may control due to the buckling of the compression flange. There are three reasons why an effective length of one can be safely assumed. First, the skinplate assembly so restrains the ribs that the usual case of cantilever buckling cannot take place. Secondly, the loading in this case produces a moment area much smaller than is associated with the normal cantilever problem. Thirdly, the 1963 American Institute of Steel Construction building code does not require an effective length of two for a cantilever beam in buckling.

The author has developed a computer program that solves the complete load, shear and moment diagrams of a rib. Four different loading conditions are usually investigated. This program is written in General Electric WIZ language for use on the G.E. 225 digital computer. This program takes less than two minutes of machine time to run. Since the best solution can only be obtained from a trial approach, this program has been a great aid in the design of both Broken Bow and Marion spillway gates.

2-2 Structural Steel Anchorage

The general approach to the problem of anchorages has been to attach the trunnion yokes to a large horizontal steel thrust girder. This girder was in turn attached to tension ties which ran through the pier to an anchorage. These ties were made of both rolled structural shapes and bars. They were insulated from the piers by a layer of soft compressible material. In some designs, steel columns inside the piers were used to take the vertical loads through the piers and into the dam. These anchorages had associated with them many problems. First, the uncertainty of design in that the unbalanced loading condition caused bending in the ties. The long ties could have considerable elongation during the loading cycle. This caused problems of gate operation and leakage. Maintenance of the ties was not possible. Corrosion and rusting of the embedded ties were possible. Freezing of water around the anchorages was also possible. Since the anchorage ties were encased in a compressible material, the anchorage could have an undesirable lateral movement under unbalanced loads. Tension ties between the trunnion assemblies were proposed to alleviate this condition.

2-3 Keystone and Eufaula Anchorages

When the Tulsa District Corps of Engineers designed Keystone and Eufaula Dams, they had available the plans of two prestressed anchorages, Eagle Gorge and Greenup (see Figure 2-1) (5). The Keystone and Eufaula designs were accomplished simultaneously and are practically identical except for some minor differences due to gate size and trunnion location. The arrangement was devised to accomplish as much of the following as possible:

(a) Transmit the maximum amount possible of the horizontal, vertical, and lateral gate reactions directly to the concrete.

(b) Minimize the variation of the concrete

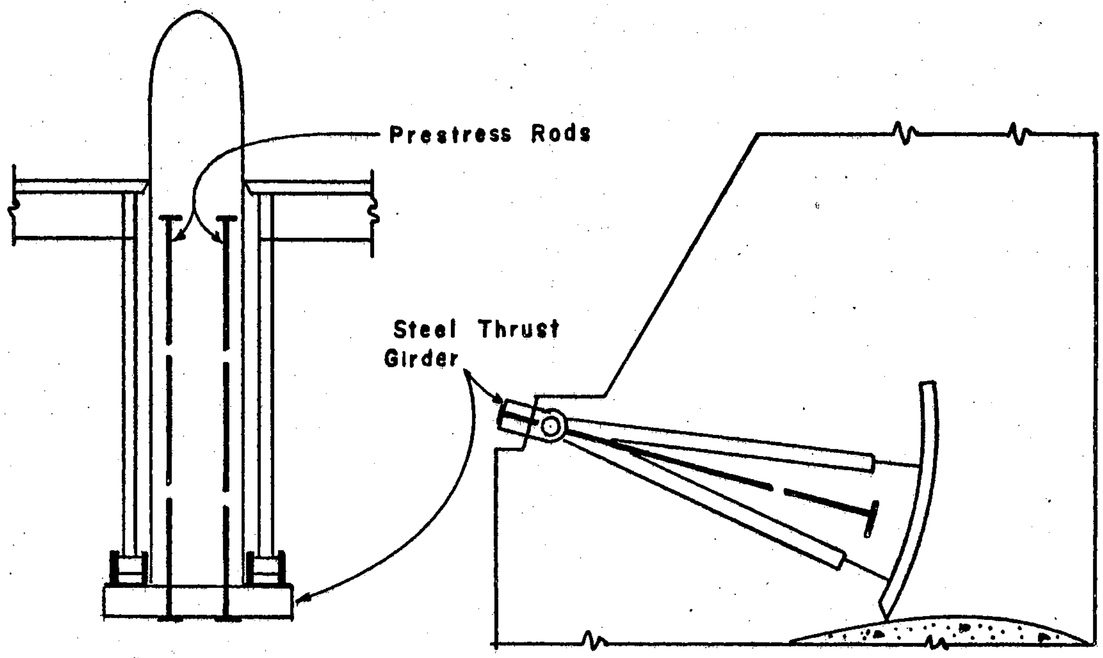


Figure 2-1. Greenup Anchorage

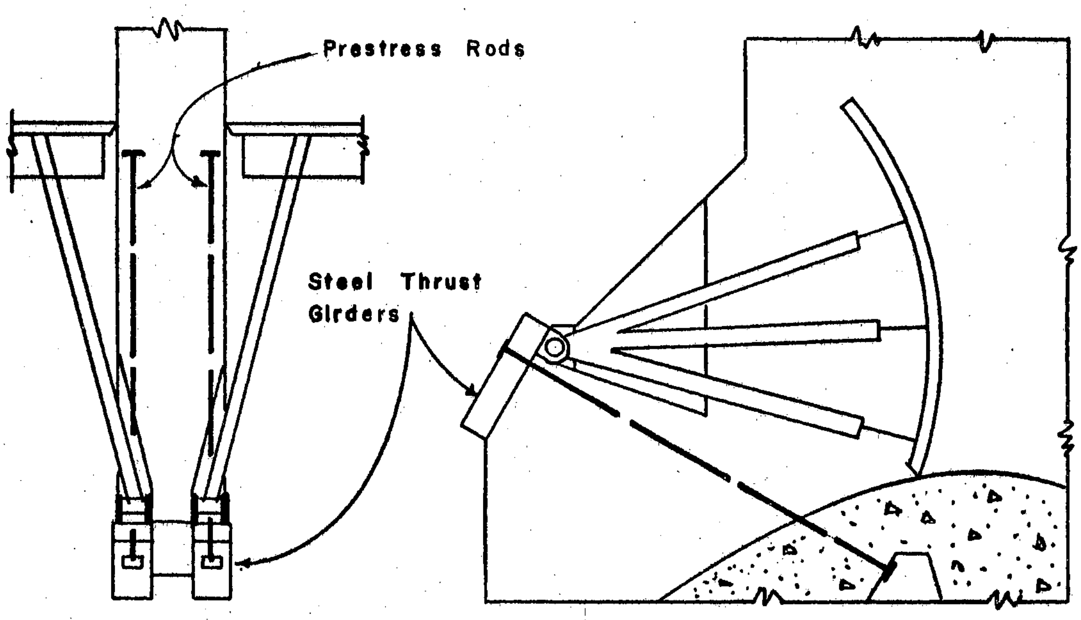


Figure 2-2. Keystone Anchorage

stress resulting from unequal loading of adjacent gates.

The selection of individual thrust girders sloped at 30° from the vertical accomplished these objectives. It had the further advantage that the anchor ends of the rods were below the weir line and could be grouted individually through an access from the ceiling of the sluice operating gallery (Figure 2-2).

Disadvantages of this design were as follows: The inclined prestressing bars went through several pours of concrete. This caused the pier concrete to have a stepped pouring sequence. The thrust girders had to be supported on a frame until the pier concrete was placed against them.

2-4 John Day Anchorage

John Day lock and dam was designed by the Walla Walla District, Corps of Engineers. It has the same configuration as the Keystone and Eufaula designs with the exception of a kick plate at the bottom of the thrust girder. This kick plate gives the thrust girder more leverage against the yoke and reduces the number of prestressing bars. The thrust girder has more moment induced in it and, therefore, has to be built heavier. The concrete bearing stress under the kick plate is higher and more localized than the concrete stress under the Keystone thrust girder.

2-5 John Redmond Anchorage

The John Redmond dam was designed for the Tulsa District by the Savannah District, Corps of Engineers. In this anchorage design a horizontal steel thrust girder and horizontal prestress bars were used. This design is similar to the Greenup design (Figure 2-3).

2-6 Wanapum Anchorage

In this design by Harza Engineering Company, Chicago, Illinois, the trunnion beam has been eliminated. The concrete taking the thrust behind the trunnion has been placed in compression by the prestressing forces (Figure 2-4) (6)(7).

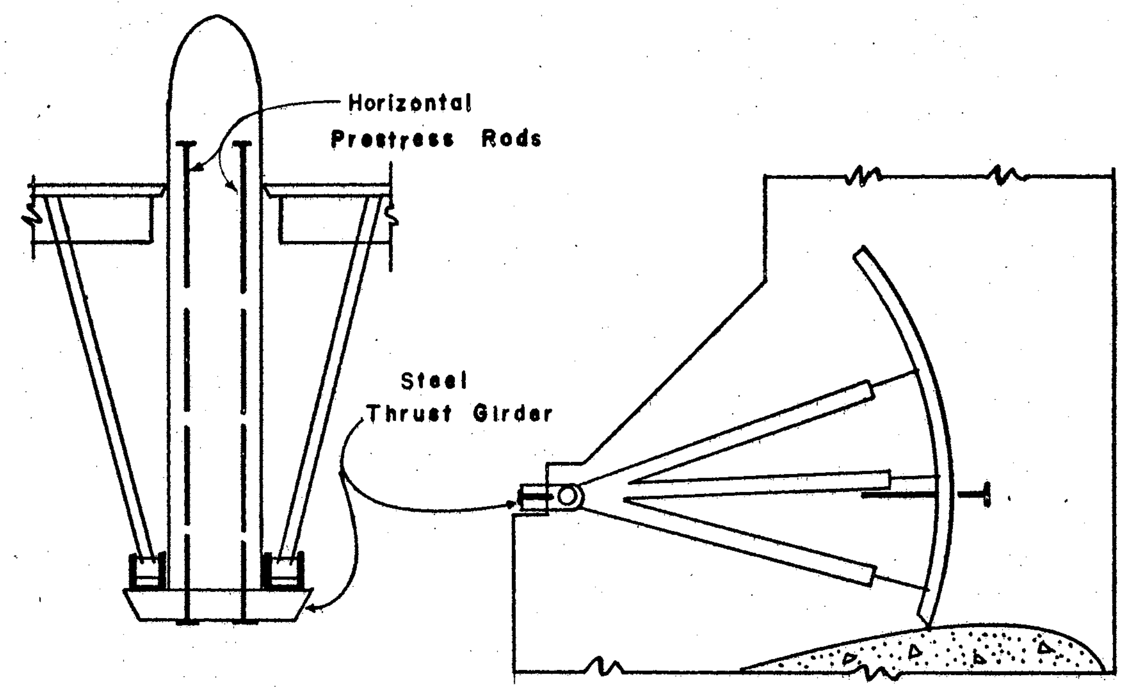


Figure 2-3. John Redmond Anchorage

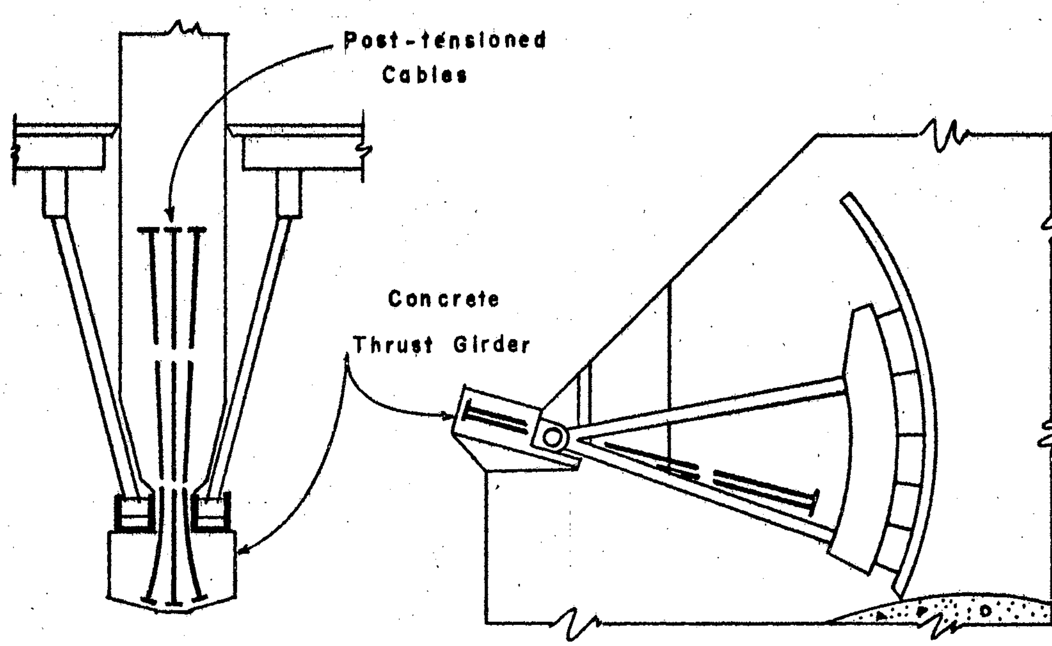


Figure 2-4. Wanapum Anchorage

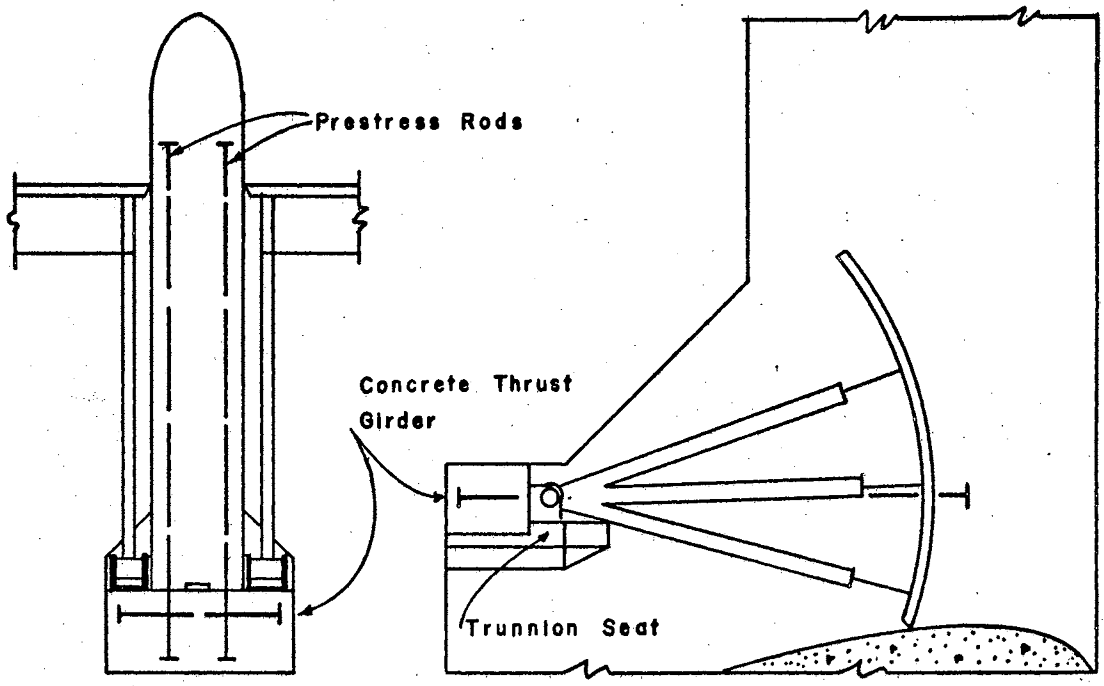


Figure 2-5. Millwood Anchorage

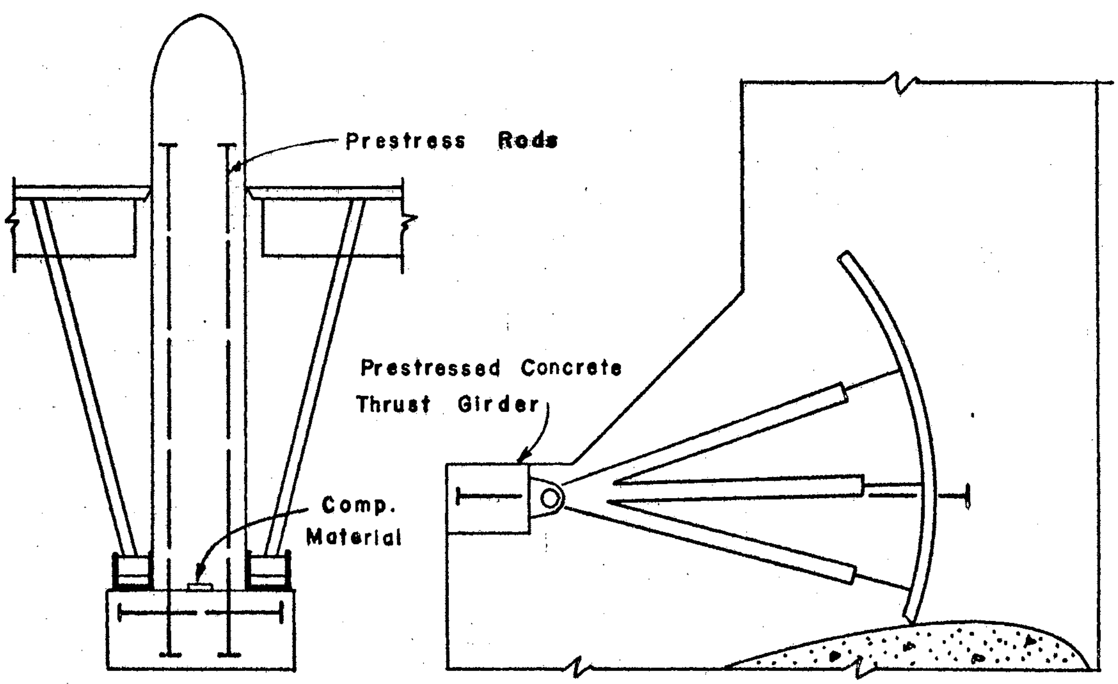


Figure 2-6. Lower Monumental Anchorage

CHAPTER III

MILLWOOD ANCHORAGE

3-1 Design Considerations

In December, 1960, Structural Section "B" was starting the preparation of the design for Millwood Dam. Several unusual factors were present in this spillway design. The lake formed by the dam will cover a flooded area of 95,200 acres. A large part of this area is heavily forested flat land. The salable timber is being harvested in the reservoir area. There is anticipated to be a large amount of logs and debris passing the dam. During a flood, the tailwater against the dam will be unusually high. At this time, the ideal trunnion location will be submerged by approximately seventeen feet. These considerations dictated the following design criteria:

(1) The tainter gates would have side frames parallel to the piers and would be armored against floating debris.

(2) The trunnion anchorage would be some form of a prestressed anchorage with the minimum amount of exposed steel surfaces and as streamlined as possible to the flow of water.

This assignment for a design was given to the author by L. E. Brown with the suggestion of using prestressed concrete as the thrust girder. The possibility of using curved or diagonal tendons to take the loads was to be investigated.

3-2 Design Criteria

After a study of several possibilities, the following conclusions were made.

To compensate for the heavier gate design of parallel side frames, ASTM A-36 steel with a basic working stress of 20,000 psi was to be used in the gates. These gates were the first to be designed by Tulsa District using this criteria.

Since the end frames were to be parallel to the pier, any thrust girder would be a short cantilever. The main consideration of this cantilever would be the shear strength as measured by the diagonal tension. The state of stress in a homogeneous material formula would be used to compute the diagonal tension. The allowable value of the diagonal tension would be limited to 0.03 of the 28 day compressive strength of the concrete. Any prestressing force would be so designed to limit the diagonal tension to this value.

The tendons which would run into the pier (parallel to flow) should be horizontal and contained in one pour of the pier. Further study showed that a concrete trunnion yoke seat, conventionally reinforced, was desirable for both ease of gate erection and to remove any torsional loads from the thrust girder.

3-3 Design Proposal

The above criteria were developed into the following proposal submitted in the Millwood Dam, Design Memorandum No. 10, 27 September 1961.

The trunnion yoke was to be set in the corner, the floor to be the concrete seat, the side wall to be the face of the pier and the back wall to be the face of a horizontal prestressed concrete thrust girder. The 36 one and one-eighth inch diameter prestressing bars in this girder were selected and located by the amount of force required to keep the diagonal tension below 150 psi at any section. The thrust girder was to be prestressed to the pier by 50 one and one-eighth inch diameter bars concentrated near each face of the pier. These bars were selected and arranged for the condition of positive "bearing" between the thrust girder and the pier with one gate loaded and the adjacent gate unloaded. The entire assembly of pier, yoke seat, and thrust girder was to be poured monolithically of 5,000 psi concrete.

Since no information was available on a prestressed concrete use of this type and magnitude, it was felt that a very conservative design should be prepared. In the first calculations for both shear of the beam and bearing against the pier, a smaller beam than actually used was investigated. And since the structure was to be monolithic with the uncertainties of the stress distributions, an extra row of bars in each direction was added to the rod patterns previously investigated. This gave the resulting rod patterns a slight eccentricity with the trunnion yoke but, due to the mass of the structure, it presented no problems.

CHAPTER IV

DEVELOPMENT

4-1 Development of Lower Monumental Anchorage

At the same time that Tulsa was developing the Millwood anchorage, Seattle District Office, Corps of Engineers, was also trying to develop a prestressed anchorage for use on Lower Monumental Dam. The first design was disapproved by the Office, Chief of Engineers, with a suggestion to try a design that was very similar to the Millwood design. This design proposed by Keith O'Donnell, Structural Branch, Office, Chief of Engineers, was worked out completely independent of the Tulsa design.

The Lower Monumental thrust girder design differed from the original Millwood design in only three features (see Figures 2-5 and 2-6, page 19).

(1) The concrete seat was not used under the trunnion yoke. Instead, the yoke was designed to take the vertical component of the load in shear to the thrust girder. This also induced a torsional load on the thrust girder.

(2) The thrust girder was not cast monolithic

with the pier. This made possible a more positive design analysis.

(3) A compressible material was used between a center portion of the pier and the thrust girder. This portion of the pier is ineffective in resisting the moment produced by the unbalanced loads. Inclusion of a compressible material raises the remaining bearing stress faster than the moment capacity is reduced, giving a net gain. The use of this compressible material is further explained in the following chapter.

4-2 Lower Monumental Model Tests

Since the gates on Lower Monumental were as large as any the Corps of Engineers had ever constructed and the maximum horizontal trunnion reaction was 2945 kips, a program of model testing was to be carried out. Dr. Arthur Anderson, of Tacoma, Washington, was retained to build and test models of the design. Dr. Anderson, besides being a consulting engineer, owns and manages a prestressed concrete plant in Tacoma.

The interim report of Dr. Anderson stated the following results:

(1) The Lower Monumental design was adequate for the proposed loads.

(2) The use of a thrust girder not bonded to the

pier is not as capable of carrying loads as one that is bonded to the pier. The shear at the interface between the pier and the bottom of the thrust girder adds significant strength to the structure.

(3) The use of the compressible material in the vertical interface was a desirable feature of the design.

4-3 Changes in Millwood Design

Following conferences with R. P. Hobson, Chief, Structural Branch, Office, Chief of Engineers, on 14 November and 28, 29 November 1962, the following changes were made in the original Millwood design.

(1) The rod patterns would be centered on the centerline of the trunnion.

(2) The thrust girder would be 6' 0" deep (vertically) and centered on the trunnion. The bottom of the girder would not be bonded to the pier.

(3) Ten prestressing bars were eliminated from the thrust girder.

With the exception of the use of the compressible material and the trunnion yoke seat, the Millwood and Lower Monumental designs were alike.

A conference was held in Seattle on 22, 23 January 1963 to present the interim report of Dr. Anderson on the first model tests. From this report, it was determined that the anchorage designed for Millwood was more than adequate to carry the anticipated loads.

In February, 1963, the Millwood design was again changed to include the compressible material. With this change, ten prestress bars 46' 9" long were eliminated per pier. This resulted in an estimated saving of about $10,000 on the project.

4-4 Other Designs

Using the experience gained from the design of the Millwood anchorage and following the directives of the Office, Chief of Engineers, the Tulsa office has completed two additional prestressed concrete anchorage designs. Each of the designs is similar to the final Millwood design with the exception of the use of a one and one-fourth inch diameter prestressing bars instead of one and one-eighth diameter bars used in Millwood. These anchorages are for Broken Bow and Pine Creek Dams. The author assisted in both of these designs.

The Nashville District Office has also designed a prestressed concrete anchorage. This anchorage to be used on J. Percy Priest Dam is of the Lower Monumental type. The maximum vertical component of the loads is 332 kips. This load causes a large amount of torsional shear on the thrust girder which would have been eliminated had the concrete yoke seat been used.

4-5 Summary of Prestressed Concrete Anchorages

Table III is a summary of all the prestressed concrete anchorages designed to this date. The dimensions have been taken from either the contract plans or the design computations. Following is an explanation of items in the table. The symbols are the same as used in the sample calculations in Chapter V.

Final prestress force in kips. This value does not exceed 60% of the minimum guaranteed strength of the bars. The minimum strength is 145,000 psi.

Eccentricity from center line. This dimension is to the center of gravity of all transverse prestress bars and is measured upstream.

Average transverse prestress. The final prestress force divided by the cross sectional area of the thrust girder.

Bearing area in square inches. The area of bearing between the thrust girder and the downstream face of the pier.

Average bearing stress. The final longitudinal prestress force divided by the bearing area.

Gate load per side. The maximum horizontal load component acting at one trunnion.

TABLE III

PRESTRESSED CONCRETE ANCHORAGES FOR INTERMEDIATE PIERS

	Millwood	Broken Bow	Rine Creek	Lower Monumental	J. Percy Priest
Thrust Girder					
Width (horizontal)	6'-0"	7'-9"	9'-0"	8'-0"	6'-0"
Depth (vertical)	6'-0"	6'-0"	9'-0"	8'-0"	5'-0"
Length	13'-6"	13'-6"	17'-6"	23'-2"	17'-2"
Transverse Prestress Force					
Number of bars	26	26	46	48	28
Diameter of bars	1 1/8"	1 1/4"	1 1/4"	1 1/4"	1 1/4"
Final force (kips)	2249	2774	4908	5000	2990
Eccentricity from ₵	9.40"	9.81"	13.59"	11.33"	8.25"
Average prestress (psi)	434	414	421	543	692
Longitudinal Prestress Force					
Number of bars	40	40	80	84	48
Diameter of bars	1 1/8"	1 1/4"	1 1/4"	1 1/4"	1 1/4"
Final force (kips)	3460	4268	8536	9000	5120
Width of pier	8'-0"	8'-0"	10'-0"	14'-0"	11'-0"
Width of compressive material	2'-0"	2'-0"	3'-0"	6'-0"	4'-4"
Bearing area (sq.in.)	4752	5184	8424	9216	4256
Average bearing stress (psi)	728	823	1013	977	1203
Gate Load Per Side (kips)	863	1092	2021	2945	1223
Average shear stress	166	163	173	320	283
Principal tensile stress	114	101	124	279	202
Bearing stress both gates	365	402	533	337	628
Bearing stresses one gate	1125	1287	1495	1400	1653
	32(T)	61(T)	51	86(T)	176

Average shear stress. The gate load per side divided by the cross sectional area of the thrust girder. There is actually a section of maximum shear slightly higher than the gate load. This shear stress does not include any consideration of torsion in the thrust girder due to the vertical load component.

Principal tensile stress. This stress is for a point on the vertical center line of the thrust girder and in the plane of the pier face. It was calculated from the shear stress of 1.5 times the average shear stress and the average transverse prestress.

Bearing stress both gates. The bearing stress remaining between the thrust girder and the pier with the application of both gates loaded.

Bearing stresses one gate. The bearing stresses at each side of the pier, between the thrust girder and the pier, due to the application of one gate load only.

Table III shows that all the designs have similar values. The significant fact shown by the table is the principal tensile stresses. The maximum principal tensile stresses within the thrust girder are slightly higher than these values. The J. Percy Priest design has a principal tensile stress of 202 psi. To have reduced this value to

150 psi, a thrust girder 6' 0" deep by 6' 6" wide with the same number of transverse bars should have been used. This would have eliminated the conventional shear reinforcing used and would also have reduced the bearing stress. This design would have increased the cost by only $200 per intermediate pier.

Lower Monumental thrust girders were designed for 6,000 psi concrete. To have reduced the principal tensile stress from 279 to 180 psi, a thrust girder with a cross sectional area of 100 square feet and the same number of transverse prestress bars would have been required. This would have increased the cost by $1,200 per intermediate pier. The same results could have been obtained with a thrust girder of 90 square feet cross sectional area and 60 one and one-fourth inch transverse prestress bars. The cost increase for this design would have been $1,500 per intermediate pier.

Another reason for using larger thrust girders is to increase the vertical distance between prestressing pipes. On Lower Monumental, a vertical spacing of five and one-half inches was used and J. Percy Priest had 6 inch spacing. The designs by the Tulsa office used nine inch vertical spacing. This larger spacing has the following advantages:

(1) More clearance between longitudinal and transverse prestressing pipes.

(2) Use of easier to install individual rod

anchor plates.

(3) Room to install reinforcing anchors to attach the protective concrete cover to the thrust girder.

CHAPTER V

SAMPLE DESIGN

5-1 Use of Compressible Material

When a tainter gate is fully raised it will have a very small horizontal trunnion reaction. Depending on the location of the hoist machinery, this reaction could even be negative (tension). In studying the case of one tainter gate fully loaded (closed) and the adjacent gate raised, it is seen that a large moment is induced across the downstream face of the pier. The load also causes tension in the pier. When the thrust girder is prestressed to the pier it must be designed to resist this loading condition.

From a beam analysis, one would expect the moment to produce a straight line stress variation across the face of the pier. If the prestressing force were sufficiently large and the pier width had been carefully selected, the pier concrete would fulfill the usual prestressed concrete requirements of no tension or excessive compression in the extreme fibers. This desired condition for the edges of the pier is no tension between the pier and the thrust girder on one side and a compressive stress not to exceed $0.35\ f'_c$ on the other side of the pier.

From strength of materials, it is known that the center of any beam does very little to resist the applied moments. This also holds true in this case of the downstream face of the pier. If a center portion of the pier face were removed from bearing with the thrust girder, the stress due to the moment would increase only slightly. Since the prestressing force causes a direct compressive stress across this face, this stress would be considerably higher. When the two stresses are added together, it will be seen that higher compressive stresses result over the entire face. Therefore, for the same gate loads, a smaller prestressing force can be used and still not have tension between the pier and thrust girder. This results in a substantial economy in the anchorage design.

Several methods of removing the pier from bearing on the thrust girder were considered. One way is to form a void between the faces. The method chosen for Millwood was to use a one inch thick soft fiber board. This performs the task of removing the center area from bearing and provides no concrete forming problems. A soft mastic material may be poured on the top of this board and seal the area from water intrusion.

5-2 Design Procedure

This section will give the detailed design procedure of a prestressed concrete trunnion anchorage.

After a gate has been designed and the complete trunnion reactions are known, the anchorage can be designed. Basically, this is a trial procedure. However, examining Table III (page 29) gives some good guide lines. If the gate size and horizontal trunnion reaction would be similar to one of the five anchorages already designed, a good idea of the anchorage is available or could be readily interpolated.

Two loading conditions must be computed. Case I is the symmetrical loading case of both gates fully loaded. Case II is for one gate loaded and the adjacent gate raised.

The amount of total longitudinal prestress force into the pier may be estimated at four times one horizontal trunnion reaction. With the size of longitudinal prestressing bars selected, the estimated required prestressing force can be adjusted to be the force supplied by an even number of bars.

The width of the void or compressible material should be selected as 25 to 40 per cent of the total pier width. The pier width is generally determined from the gate size. If a pier width is selected from Table III, no trouble should be encountered in its design. Assuming either uniform bearing (Case I) between the pier and thrust girder or straight line variation (Case II), the load, shear, and moment diagrams of the thrust girder can be computed and drawn. If these diagrams are kept in terms of kips, kips per foot, and kip feet, the cross sectional dimensions of the thrust girder are immaterial.

The vertical depth of the thrust girder is next tentatively selected. The maximum bearing stress can be obtained by dividing the maximum bearing load by the tentative vertical depth of the thrust girder. This value must be less than 0.35 f'_c to be satisfactory. The depth of the beam must also be great enough to permit the number of required horizontal rows of prestressed bars to be placed in the thrust girder. It is suggested that a dimension of six to nine inches be used for the vertical spacing of the bars. The author prefers the larger spacing which allows more clearance between the mutually perpendicular prestressing pipe chases.

The horizontal spacing of the bars can be from four and one-half or five inches up. The center of the first bar should not be closer to the face of the pier than about eight inches. This allows only room for the required reinforcing and the necessary concrete clearance. On Lower Monumental Dam, the first row of bars was one foot from the face of the pier and the others were spaced at one foot horizontally.

The horizontal dimension of the thrust girder can be the same or larger than the vertical dimension. One way to select this value is to calculate the required area necessary to keep the simple shear stress of the thrust girder below some value. This value will probably be between 175 and 200 psi. This low value of simple shear will give a minimum required number of transverse prestress bars. A study between the number of bars and amount of concrete could determine a minimum cost.

The section of maximum shear will occur at the edge of the longitudinal prestress bars. Assuming a parabolic shear stress distribution across the thrust girder will give the maximum shear equal to one and one-half times the average shear and will occur at the center of the girder. When this value of shear is combined with the transverse prestress, the principal tensile stress must not exceed $0.03\ f'_c$. The state of stress of a homogeneous material equation can be solved for the required amount of transverse prestress to meet this condition. If a sufficient number of bars are furnished to give this required transverse prestress over the entire cross section of the thrust girder, the value of the allowable principal tensile stress will have been met. This calculation will not change when the eccentricity of the transverse rods are changed. An even number of transverse prestress bars should be chosen.

The amount of eccentricity of the transverse rods is determined from two requirements. First, the center of gravity of the rods must lie slightly within the kern so that the downstream edge of the thrust girder will always be in compression (100 psi minimum). Secondly, the eccentricity must be far enough upstream so that at the section of maximum moment of the thrust girder the upstream edge will be in compression. Numerous bar arrangements may be devised, all having the same center of gravity. The bars should be spread over as much of the thrust girder as possible. A few bars near the downstream edge is not difficult to obtain.

The jacking sequence should be so designed that the stress level over the entire thrust girder cross section is always increased and never decreased. The rods may be jacked using either two or four jacks simultaneously. Symmetry of jacking should be maintained about the horizontal center line.

For the jacking sequence of the longitudinal bars, either two or four jacks may be used. The jacking sequence must be symmetrical about both the horizontal and vertical center lines. The rods nearest the pier faces are generally jacked latest in the sequence.

No conventional shear reinforcing is provided in the design. A cage of reinforcing steel bars has been provided at each end and at the center of the thrust girder. A mat of reinforcing bars is also used in the downstream face of the pier next to the thrust girder. At the upstream anchor end of the longitudinal bars, a reinforcing cage is used. Each of these reinforcing cages is designed to resist four per cent of the total prestress force normal to that face and should be adequate to resist any bursting stresses.

5-3 Sample Calculations

The design example used in this section is similar to the Millwood Dam trunnion anchorage. The contract drawings for this project are given in the Appendix.

The following information is given at the start of this design:

Maximum horizontal trunnion reaction R_H = 863.22 kips

Nominal pier width 8' - 0"

$1^1/8$" diameter prestress rods

Initial prestress force per rod 101.0 kips

Final prestress force per rod F = 86.5 kips

The 28 day compressive strength of concrete f'_c = 5,000 psi.

Maximum allowable principal tensile stress is S_n = 150 psi.

Computations

$4R_H$ = 4 x 863.22 = 3,453 kips.

Approximate number of rods =

3,453/86.5 = 39.9 rods.

Use 40 rods

Final longitudinal prestressing force

$$F_1 = 40F = 40 \times 86.5 = 3{,}460 \text{ kips.}$$

Case I - Symmetrical Loading

Average trunnion yoke load = 863.22/2.67'

= 323.3 kips/ft.

Average Prestress load = 1,730/2.0'

= 865 kips/ft.

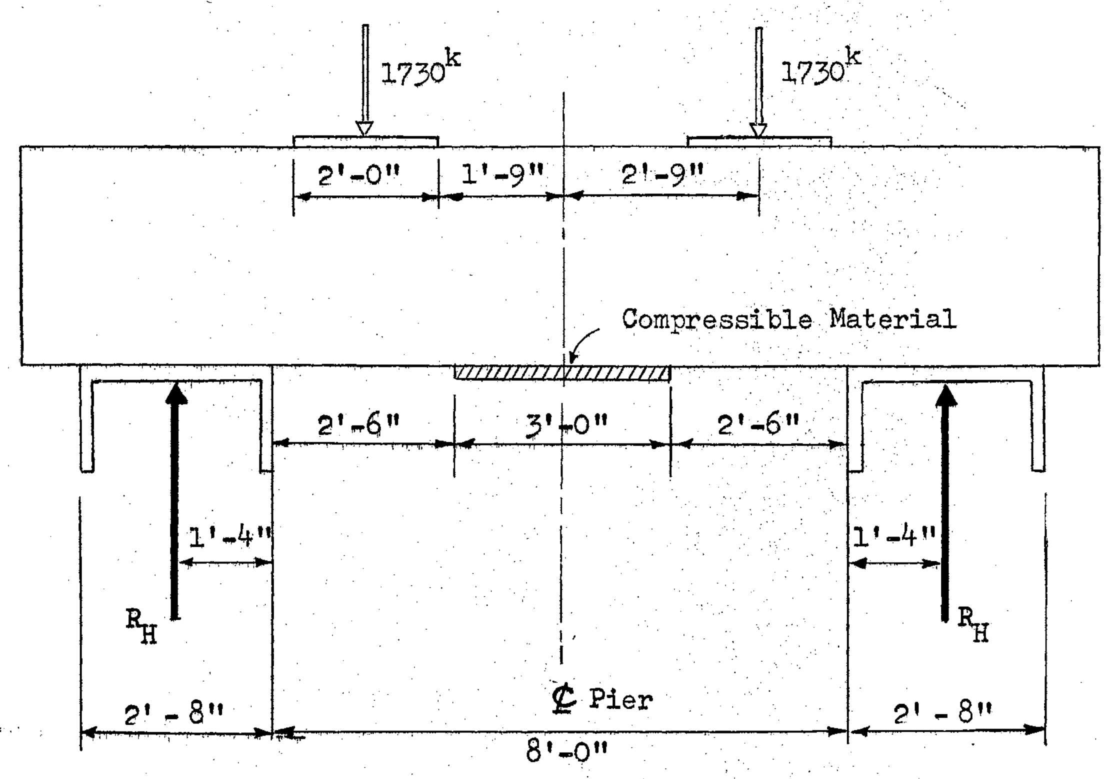


Figure 5-1. Thrust Girder, Plan View

Case I - Load, Shear and Moment Diagrams

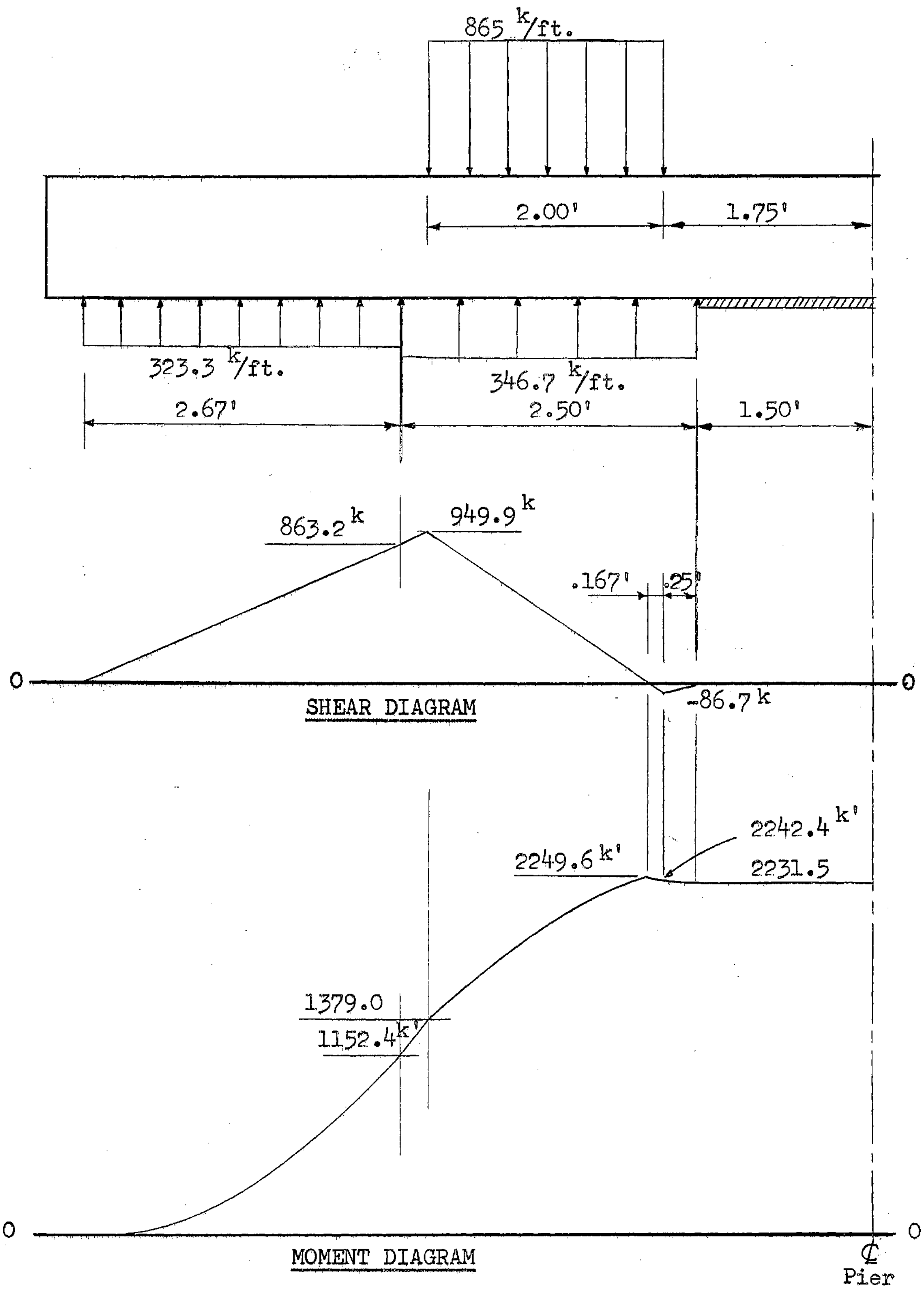


Figure 5-2. Load, Shear, Moment Diagram, Case I

Case II - Unsymmetrical Loading

Net normal force $= F_1 - R_H = 3{,}460 - 863.22$

$= 2{,}596.78$ kips.

Bearing Area = 5' long

Moment $= R_H e' = 863.22 \times 5.33' = 4{,}600.96$ kip ft.

Section modulus $= S_{pier} = \frac{1}{6}(8^2 - 3^2) = 9.1667$ ft².

Maximum and minimum bearing pressures

$$f_{bng} = \frac{F_1 - R_H}{A_{bng}} \pm \frac{R_H e'}{S_{pier}} = 2{,}596.78/5 \pm 4{,}600.96/9.1667$$

$= 519.36 \pm 501.92$

$= 1{,}021.28$ and 17.44 kips/ft.

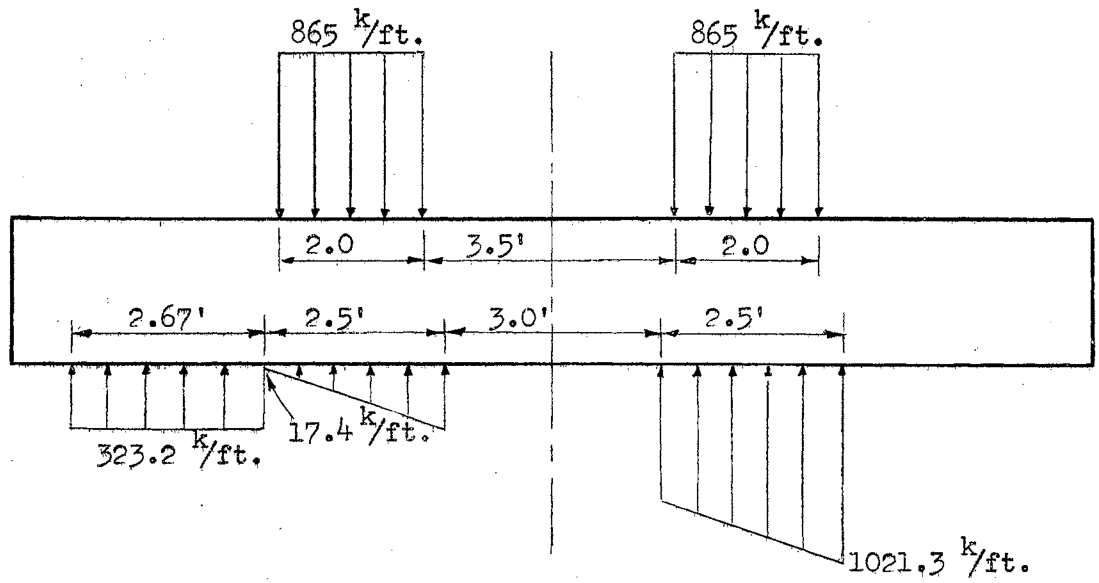


Figure 5-3. Load Diagram, Case II

From this loading diagram, it is seen that the maximum shear and moments from Case I will always be greater than from Case II.

Try a 6' - 0" deep thrust girder for maximum bearing stress.

$$f_{bng} = \frac{1{,}021.3 \text{ k/ft.}}{6.0' \times 144} = 1{,}182 \text{ psi} < 1{,}750 \text{ psi.}$$

Minimum bearing stress

$$f_{bng} = \frac{17.4 \text{ k/ft.}}{6.0' \times 144} = 20 \text{ psi} > 0 \text{ .}$$

Now, select the horizontal dimension of the thrust girder. The desired average shear stress is 190 psi. The maximum shear is 949.9 kips.

$$\frac{949{,}000 \text{ lb.}}{6.0 \times 190 \times 144} = 5.8 \text{ ft.}$$

Use a thrust girder 6' - 0" vertically by 6' - 0" horizontally.

Maximum average shear stress

$$v_{avg} = \frac{949{,}000 \text{ lb.}}{6.0' \times 6.0' \times 144} = 183 \text{ psi.}$$

Maximum shear stress

$$v_{max} = 1.5 \times 183 = 275 \text{ psi.}$$

Required transverse prestress so that the principal tensile stress does not exceed 150 psi.

$$f_t = \frac{(v_{max})^2}{150} - 150 = \frac{(275)^2}{150} - 150 = 354 \text{ psi.}$$

Required number of transverse rods for shear stress

$$\frac{354 \text{ psi x } 6.0' \text{ x } 6.0' \text{ x } 144}{86.5 \text{ kips/rod}} = 21.2 \text{ rods.}$$

Maximum bending moment in thrust girder is 2,249.6 kip-feet. The final eccentricity of transverse prestress rods has to be less than 1.0 feet.

To find the required number of rods to resist the moment.

Thrust girder section modulus

$$S_{TG} = \frac{6.0 \text{ x } 6.0^2}{6} = 36 \text{ ft}^3 .$$

Thrust girder cross sectional area

$$A_t = 6.0 \text{ x } 6.0 = 36 \text{ ft}^2 .$$

To find the minimum number of bars required to give a residual compressive stress of 100 psi or approximately 15 kips per square foot at the upstream face of the thrust girder.

$$N_t = \frac{15 + {}^M\!/S}{({}^1\!/A_t + {}^e\!/S)F} = \frac{15 + {}^{2,249.6}\!/36}{({}^1\!/36 + {}^1\!/36)\,86.5}$$

$$= 17.6 \text{ rods.}$$

Since the maximum eccentricity was used, the actual number of bars required is larger than this value. From the previous calculation for shear stress, 22 bars have to be used.

Transverse prestress force

$$F_t = N_t \times F = 22 \times 86.5 \text{ kips/bar} = 1{,}903 \text{ kips.}$$

Average transverse prestress

$$f_t = \frac{F_t}{A_t} = \frac{1{,}903}{36} = 52.86 \text{ ksf or } 367 \text{ psi.}$$

For 100 psi compression at the downstream face of the thrust girder

$$100 = \frac{F_t}{A_t} - \frac{F_t e}{S_{TG}} = \frac{1{,}903}{36 \times 144} - \frac{1{,}903\ e}{36 \times 1{,}728}$$

Solving for e:

$$e = 8.73 \text{ inches maximum.}$$

Assume the transverse rod pattern selected has e = 8.70 inches. Then, the no load stresses are

$$f_t = \frac{F_t}{A_t} \pm \frac{F_t e}{S_{TG}} = \frac{1{,}903}{36 \times 144} \pm \frac{1{,}903 \times 8.70}{36 \times 1{,}728}$$

$$= 633 \text{ psi and } 101 \text{ psi.}$$

Stresses at section of maximum moment

$$f_t = \frac{F_t}{A_t} \pm \frac{F_t e}{S_{TG}} \mp \frac{M}{S_{TG}}$$

$$= \frac{1{,}903}{36 \times 144} \pm \frac{1{,}903 \times 8.70}{36 \times 1{,}728} \mp \frac{2{,}249.6}{36 \times 144}$$

= 199 psi at upstream face

and 535 psi at downstream face.

The maximum principal tensile stress:

$$S_n = \frac{-f_c}{2} + \sqrt{\left(\frac{f_c}{2}\right)^2 + v_{max}^2}$$

$$+ \frac{-367}{2} + \sqrt{\left(\frac{367}{2}\right)^2 + (275)^2}$$

$$= 147 \text{ psi} < 150 \text{ psi}.$$

CHAPTER VI

COST COMPARISONS

6-1 Abstracts of Bids

In the following abstracts of bids, the Government Estimate (G.E.) is listed first. The other bids are listed in ascending order of total bid on the complete project. The average includes bids and the Government Estimate. The median is the bid which has the same number of bids lower as higher. The Government Estimate was considered as a bid. In the cases of an even number of bids, the median was considered as the lower of the two.

6-2 Cost of Trunnion Anchorage and Embedded Metal

To compare the costs of trunnion anchorage systems, a common denominator must be found. This can be taken as the force the anchorage system must resist. This force can be expressed simply as a function of the gate width and the square of the gate height. In the abstract of bids, the trunnion anchorage has generally been lumped with an item of embedded metal. This embedded metal is the stainless steel side and sill plates. These items are proportional to twice the gate height plus the gate width.

TABLE IV

TORONTO DAM

Verdigris River, Kansas
Bids opened 4 April 1956
Total Winning bid $4,762,666.00

	43. Tainter Gate Embedded Metal and Trunnion Anchorage	44. Tainter Gates and Accessories	Total
G.E.	$44,442	$278,200	$322,642
7	55,523	207,141	262,664
3	76,450	289,450	365,900
4	73,100	239,300	312,400
High	76,450	289,450	365,900
Low	44,442	207,141	262,664
Avg.	62,379	253,523	315,902

The abstract of the three low biders only.

TABLE V

KEYSTONE DAM

Arkansas River, Oklahoma
Bids opened 10 November 1959
Total Winning bid $16,173,876.86

	34. Tainter Gate Embedded Metal and Trunnion Anchorage	35. Tainter Gates and Accessories	Total
G.E.	$323,700	$857,200	$1,180,900
12	380,390	825,000	1,205,390
2	337,000	700,000	1,037,000
3	300,000	710,000	1,010,000
11	436,800	892,400	1,329,200
5	517,000	866,000	1,383,000
13	300,000	500,000	800,000
4	325,000	720,000	1,045,000
16	485,000	1,090,700	1,575,700
10	475,000	775,000	1,250,000
14	420,000	775,000	1,195,000
7	172,306	550,000	722,306
6	417,760	815,000	1,232,760
15	300,000	500,000	800,000
9	375,000	720,000	1,095,000
8	579,000	798,000	1,377,000
1	461,539	860,081	1,321,620
17	520,000	913,000	1,433,000
High	579,000	1,090,700	1,575,700
Low	172,306	500,000	722,306
Avg.	395,861	770,410	1,166,271
Median	380,390	775,000	1,195,000

TABLE VI

EUFAULA DAM

Canadian River, Oklahoma
Bids opened 17 December 1959
Total Winning bid $14,672,709.00

	55. Tainter Gate Embedded Metal and Trunnion Anchorage	56. Tainter Gates and Accessories	Total
G.E.	$188,000	$492,000	$680,000
10	135,000	300,000	435,000
8	110,000	420,000	530,000
2	100,000	200,000	300,000
1	200,000	330,200	530,200
5	235,000	440,000	675,000
3	300,000	525,000	825,000
9	185,000	410,000	595,000
7	165,000	410,000	575,000
6	135,000	350,000	485,000
4	180,000	460,000	640,000
High	300,000	525,000	825,000
Low	100,000	200,000	300,000
Avg.	175,727	394,291	570,018
Median	180,000	410,000	575,000

TABLE VII

JOHN REDMOND DAM

Grand River, Kansas
Bids opened 2 March 1961
Total Winning bid $7,502,946.40

	58. Tainter Gate Embedded Metal and Trunnion Anchorage	59. Tainter Gates and Accessories	Total
G.E.	$160,000	$646,000	$ 806,000
11	118,500	485,000	603,500
2	130,000	495,000	625,000
12	165,000	610,000	775,000
16	330,000	608,000	938,000
3	200,000	584,000	784,000
1	270,921	735,211	1,010,132
9	205,000	583,000	788,000
14	159,443	607,455	766,898
4	185,000	550,000	735,000
6	212,300	585,300	797,600
7	163,000	555,000	718,000
13	200,000	500,000	700,000
17	149,413	550,000	699,413
8	325,000	900,000	1,225,000
10	218,000	530,000	748,000
5	185,000	577,700	762,700
High	330,000	900,000	1,225,000
Low	118,500	485,000	603,000
Avg.	198,857	594,216	793,072
Median	185,000	583,000	766,898

TABLE VIII

MILLWOOD DAM

Little River, Arkansas
Bids opened 14 March 1963
Total Winning bid $10,793,596.89

	47. Tainter Gate Embedded Metal and Trunnion Anchorage	48. Tainter Gates and Accessories	Total
G.E.	$178,900	$500,900	$679,800
6*	150,809	451,282	602,091
5	200,000	500,000	700,000
9	150,000	525,000	675,000
1	141,700	585,000	726,000
10	230,000	650,000	880,000
12	125,700	500,000	625,700
7	150,000	500,000	650,000
14	80,000	500,000	580,000
2	156,000	560,000	716,000
13	117,430	623,650	741,080
3	200,000	600,000	800,000
4	105,600	600,000	705,600
15	205,000	630,000	835,000
11	200,000	655,000	855,000
8	125,000	560,000	685,000
High	230,000	655,000	880,000
Low	80,000	451,282	580,000
Avg.	157,259	562,662	716,061
Median	150,000	560,000	700,000

*Bidder disqualified for incomplete bid.

TABLE IX

JOHN DAY LOCK AND DAM

(North Shore Construction)
Columbia River, Washington
Bids opened 10 December 1959
Total Winning bid $32,381,096.00

	145. Furnish Tainter Gate Each	146. Install Tainter Gate Each	Total Gate
G.E.	$ 91,500	$19,100	$ 110,600
1	87,500	12,000	99,500
2	75,000	16,000	91,000
3	113,500	30,800	144,300
			$ 111,350 (avg)

	147. Furnish 40 Anchorage Assem. and Appurtenances	148. Install 38 Anchorage Assem. and Appurtenances	Total Anchorage per Gate
G.E.	$710,000	$113,200	$41,458
1	550,000	100,000	32,763
2	600,000	90,000	34,737
3	479,000	172,000	33,003
			$35,490 (avg)

	152. Furnish Tainter Gate Side Seal Plates, Unheated each	153. Install Tainter Gate Side Seal Plates Unheated each	Total Side Seals per Gate
G.E.	$1,300	$890	$4,380
1	1,200	950	4,300
2	1,000	900	3,800
3	2,800	740	7,080
			$4,890 (avg)

Two side seals required per gate.

TABLE IX (Continued)

	Total Anchorage and Side Seals	Total Gate Installation per Gate*
G.E.	$45,838	$156,438
1	37,063	136,563
2	38,537	129,537
3	40,083	184,383
Average	$40,380	$151,730

*Does not include cost of sill beam.

TABLE X

LOWER MONUMENTAL DAM

(South Shore Construction)
Snake River, Washington
Bids opened 15 March 1962
Total Winning bid $24,988,656.00

	130. Furnish Spillway Gate Each	131. Install Spillway Gate Each	Total Gate
G.E.	$106,000	$21,000	$127,000
7	90,625	50,000	140,625
2	91,000	24,000	115,000
1	90,000	40,000	130,000
			$128,156 (avg)

	132. Furnish Anchorage Assem. and Appurtenances Each	133. Install Anchorage and Appurtenances Each	Total Anchorage per Gate
G.E.	$7,680	$3,875	$23,110
7	4,000	4,500	17,000
2	9,000	3,090	24,180
1	9,000	4,600	27,200
			$22,872 (avg)

	134. Furnish Gate Side Seal Plate Unheated Each	135. Install Gate Side Seal Plate Unheated Each	Total per Gate
G.E.	$1,465	$1,070	$5,070
7	500	1,000	3,000
2	1,600	1,300	5,800
1	1,500	700	4,400
			$4,568 (avg)

Two side seals required per gate.

TABLE X (Continued)

	Total Anchorage and Seals	Total Gate Installation per Gate
G.E.	$28,180	$155,180
7	20,000	160,625
2	29,980	144,980
1	31,600	161,600
Average	27,440	$155,596*

*Does not include cost of sill beam.

TABLE XI

J. PERCY PRIEST RESERVOIR

Stones River, Davidson County, Tennessee
Bids opened 28 May 1963
Total Winning bid $9,168,865.00

	45. Tainter Gate Guides and Sills	46. Tainter Gate Anchorages	47. Tainter Gates	Total 45,46	Total 45,46,47
G.E.	28,700	79,300	283,000	108,000	391,000
10	25,000	75,000	200,000	100,000	300,000
8	21,000	60,000	233,000	81,000	314,000
11	30,000	60,000	255,000	90,000	345,000
7	23,500	10,000*	300,000	33,500*	333,500
5	22,700	45,000	265,000	67,700	332,700
4	16,000	58,500	180,074	74,500	254,574
3	15,000	75,000	200,000	90,000	290,000
6	37,000	70,000	150,000	107,000	257,000
9	20,000	55,000	300,000	75,000	375,000
1	20,000	48,000	270,000	68,000	338,000
2	25,000	35,000	200,000	60,000	260,000
12	28,000	75,000	261,000	103,000	364,000
High	37,000	79,300	300.000	108,000	391,000
Low	15,000	35,000	150,000	60,000	254,574
Average	23,992	61,317	238,236	85,350	319,598
Median	23,500	60,000	255,000	81,000	332,700

*Not used as a valid bid.

Since the cost of the embedded metal is an unknown proportion of the cost of the anchorage system, the entire bid item will be related to the factor of gate width times height squared. Table XII shows the results of this computation.

From Table XII it can be seen that the cost of the Millwood anchorage was only slightly higher than the last conventional structural steel tainter gate anchorage designed by Tulsa for Toronto Dam. The J. Percy Priest Dam anchorage was slightly cheaper than the Toronto Anchorage. The Millwood anchorage cost 79% of the Eufaula anchorage and 75% of the Keystone anchorage. While the John Redmond anchorage was bid as the least expensive anchorage system of medium size gates, it had features that were thought to be undesirable for the Millwood anchorage.

The lower unit cost of both John Day and Lower Monumental Dam anchorage systems seems to indicate an economy for the larger gates. The bids for these two gates of the same size proves that the prestressed concrete trunnion anchorage of Lower Monumental Dam is cheaper than the prestressed steel beam anchorage of the John Day Dam.

6-3 Total Cost of Gates

In planning a dam, the question of economical size of gates arises. The total cost of several gates and spillway structures can be compared. In each case, the combination of gate and spillway have to be both hydraulically

TABLE XII

TRUNNION ANCHORAGE AND EMBEDDED METAL

Name	Bid	No. Gates	Misc. Cost Per Gate*	Bid Per Gate	Width x Height	Bid Per WH^2
Toronto	$ 63,379	8	----	$ 7,797.37	40x25.52	$0.29931
Keystone	$380,390	18	----	$21,132.78	40x36.19	$0.40338
Eufaula	$180,000	11	----	$16,363.64	40x32.73	$0.38188
J. Redmond	$185,000	14	----	$13,214.29	40x36.19	$0.25223
Millwood	$150,000	13	$1116	$12,654.46	40x32.39	$0.30156
John Day	$ 40,380	Ea. of 20	----	$40,380.00	50x59.95	$0.22471
Lower Monumental	$ 27,440	Ea. of 8	$1925	$29,365.00	50x60.56	$0.16014
J. Percy Priest	$ 81,000	4	$ 600	$21,100.00	45x41.43	$0.29208

*Estimated cost of concrete in thrust girder.

TABLE XIII

TOTAL GATE INSTALLATION

Name	Bid Date	Total Bid	No. Gates	Bid Per Gate	W x H	$/W x H
Toronto	4 Apr 56	$ 315,902	8	$ 39,488	40x25.52	$38.68
Keystone	10 Nov 59	$1,195,000	18	$ 66,389	40x36.19	$45.86
Eufaula	17 Dec 59	$ 575,000	11	$ 52,273	40x32.73	$39.93
John Redmond	2 Mar 61	$ 766,898	14	$ 54,778	40x36.19	$37.84
Millwood	14 Mar 63	$ 700,000	13	$ 54,962*	40x32.39	$42.42
John Day	10 Dec 59	Each	20	$151,730	50x59.95	$50.62
Lower Monu-mental	15 Mar 62	Each	8	$157,521*	50x60.56	$52.02
J. Percy Priest	28 May 63	$ 332,700	4	$ 83,775*	45x41.43	$44.94

*Includes miscellaneous cost from Table XII.

compatible and structurally stable. Only by comparing the total cost can the most economical dam be designed. Such a study was made during the planning for Gillham Spillway. The semi-logarithmic graph (Figure 6-1) was first used in this study.

The total costs of the gates were taken from the abstract of bids. The median of all bids was used where available. This value was generally close to both the average bid and the winning bid. Where only the three low bids were available, the average of the bids and the Government estimate was used.

The scattering of points (Figure 6-1) cannot be entirely explained.

Toronto anchorage is a structural steel design.

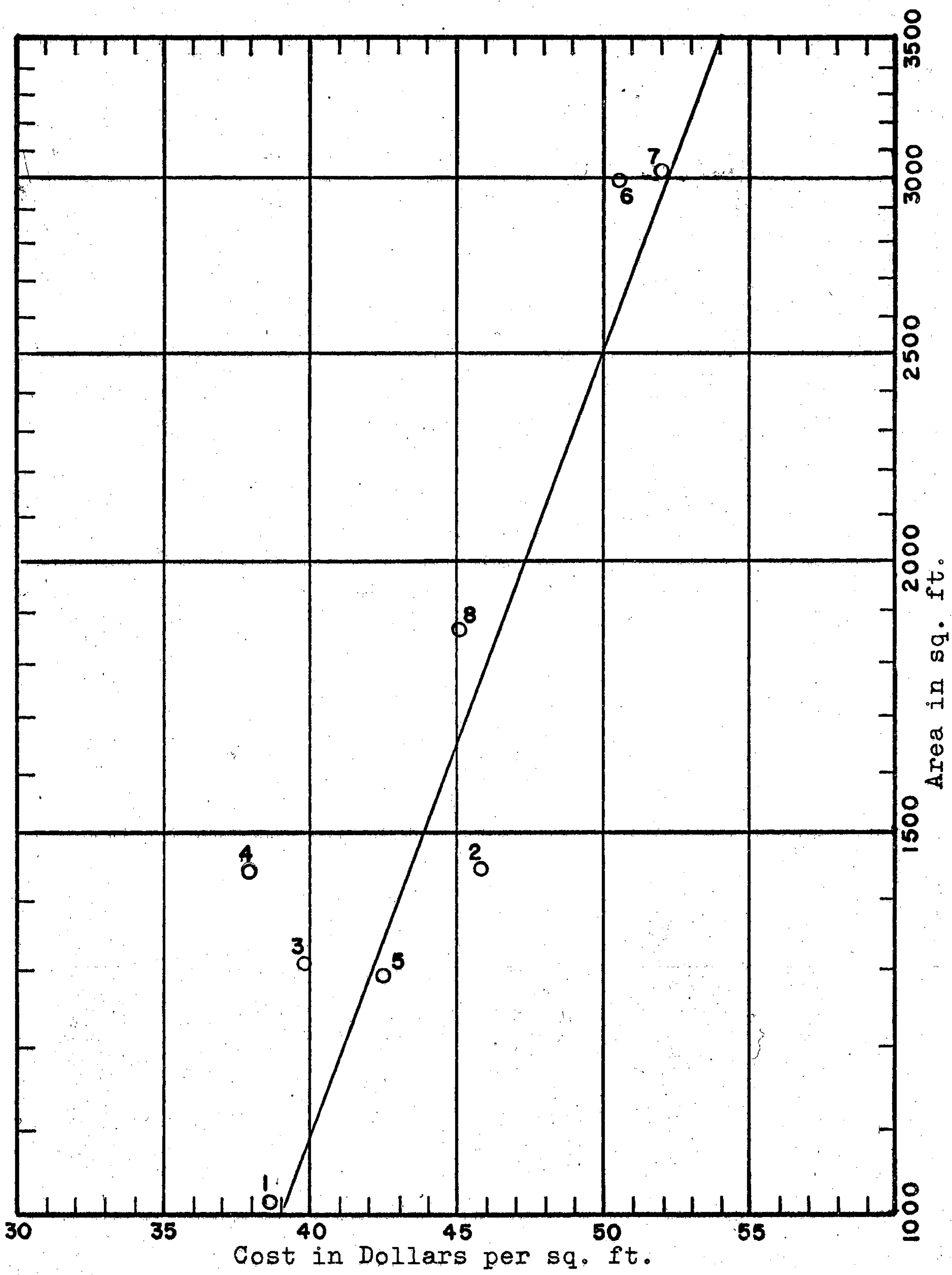
Keystone gate bids seem to be higher possibly because it was a new design and the prestressing of trunnion anchorages was not familiar to the contractors.

Eufaula, bid just one month later than Keystone, was considerably lower in unit price.

Millwood has a heavy gate due to the parallel end frames. This raises the unit cost.

J. Percy Priest has a smaller anchorage than Tulsa would design. There may also be some economy in larger gates not shown by the graph.

No price corrections have been applied to the bid figures. The basic price of steel has remained constant since mid-1958.



1. Toronto
2. Keystone
3. Eufaula
4. John Redmond
5. Millwood
6. John Day
7. Lower Monumental
8. J. Percy Priest

Figure 6-1. Tainter Gate Cost Per Square Foot

CHAPTER VII

SUMMARY AND CONCLUSIONS

Millwood Dam prestressed concrete trunnion anchorage was conceived and developed using sound basic engineering principles. The amount of construction involved in the trunnion anchorage and embedded metal items for Millwood, Broken Bow and Pine Creek dams may amount to $505,000. The final cost of the anchorages has been shown to be considerably less than the former prestressed trunnion anchorage designs used by Tulsa. Had the Eufaula Dam design and prices been extrapolated to Millwood, Broken Bow and Pine Creek Dams, the cost may have been $132,000 greater than the prestressed concrete trunnion anchorages.

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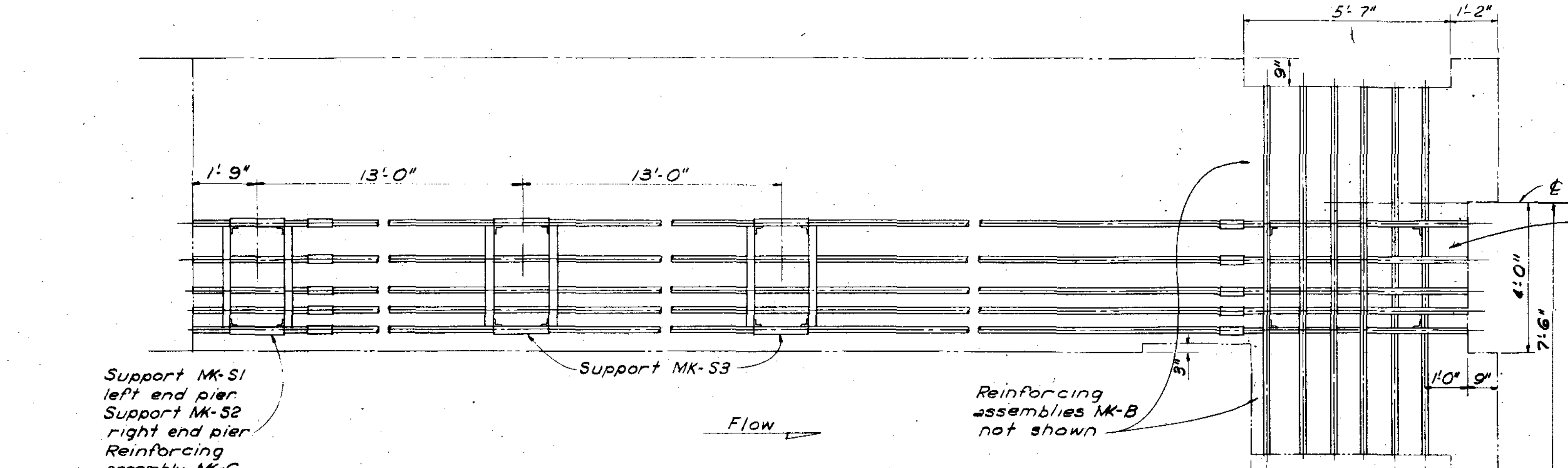
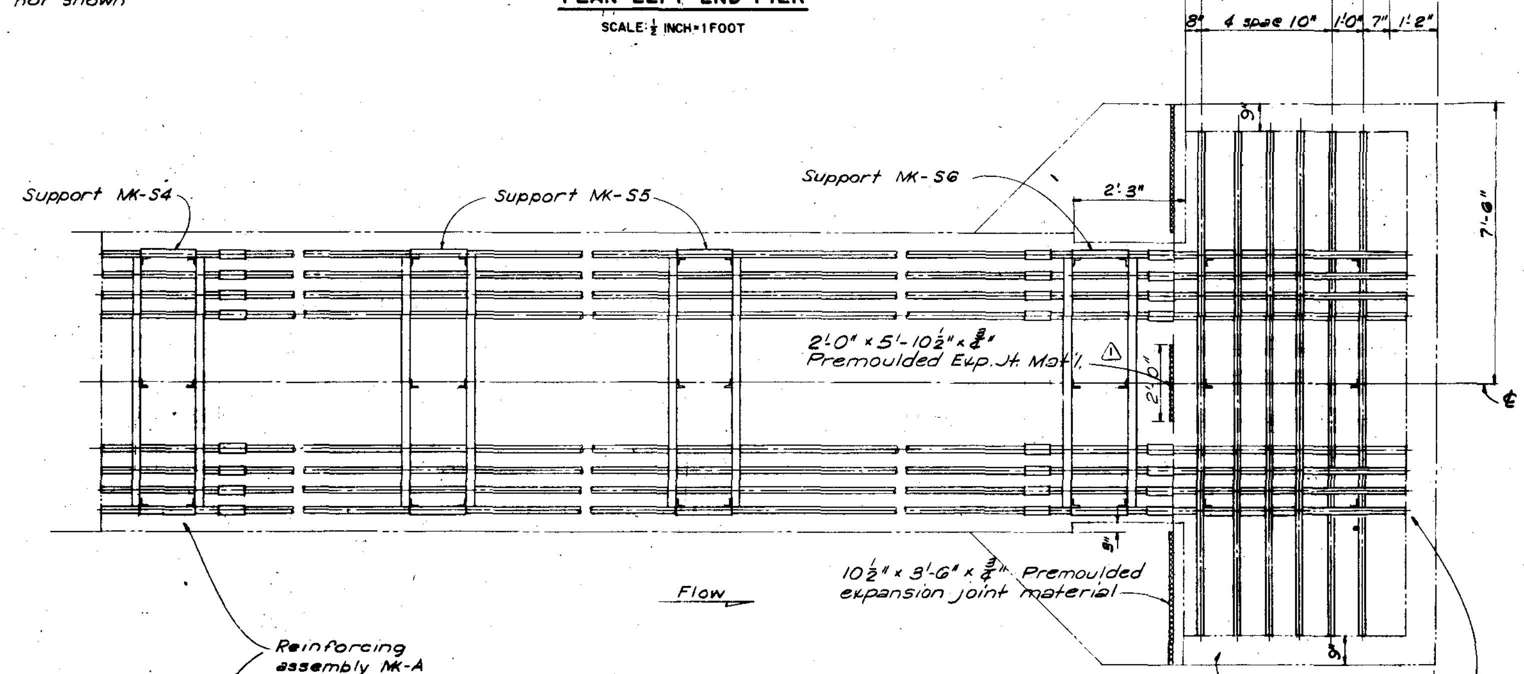
APPENDIX

The following six plates illustrate the construction drawings of a prestressed concrete trunnion anchorage. These drawings were developed for Millwood Dam. A similar set of drawings is used in the contract plans for Broken Bow Dam.



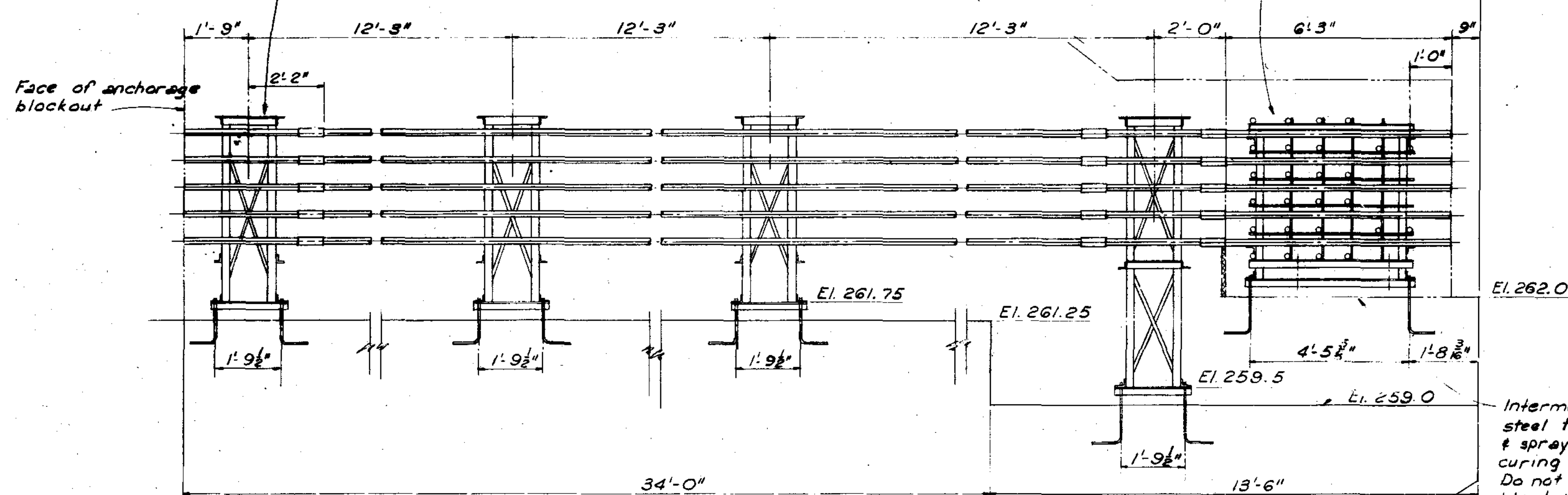
PLAN LEFT END PIER

SCALE: ½ INCH = 1 FOOT



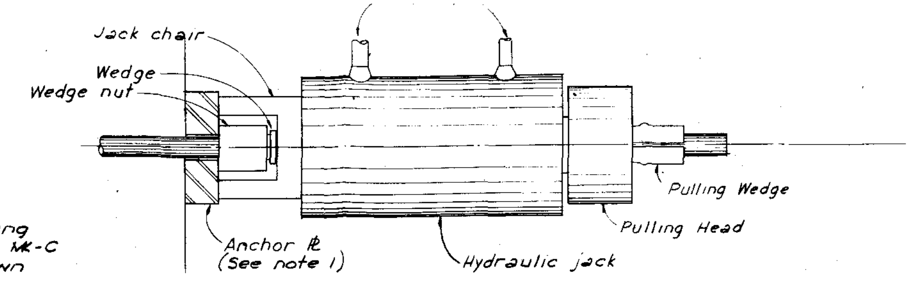
PLAN INTERMEDIATE PIER ⚠

SCALE: ½ INCH = 1 FOOT

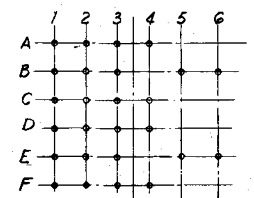


ELEVATION

SCALE: ½ INCH = 1 FOOT



ROD JACKING FACILITY

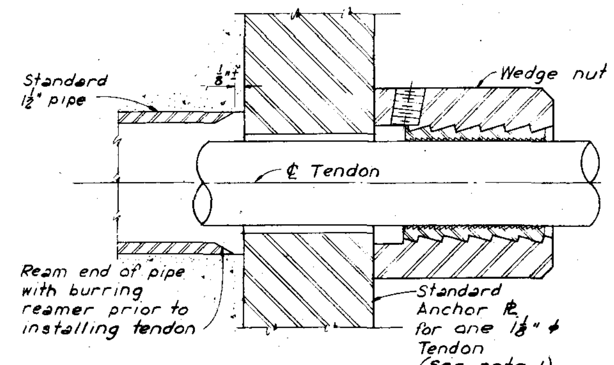


ROD PATTERN FOR SHORT RODS

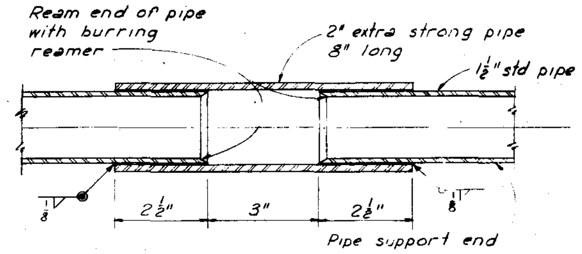
Stage	Tendons
1st	B2, B5, E2, E5
2nd	B1, B6, E1, E6
3rd	A1, A4, F1, F4
4th	A2, C3, F2, D3
5th	A3, C2, F3, D2
6th	C1, C4, D1, D4
7th	B3, E3

STRESSING SEQUENCE FOR SHORT RODS

(THRUST GIRDER)



ANCHOR DETAIL



PIPE COUPLING DETAIL

SCALE: 6 INCHES = 1 FOOT

Stage	Tendons
1st	H3, H6, J3, J6
2nd	G1, G8, K1, K8
3rd	I1, I4, I5, I8
4th	G4, G5, K4, K5
5th	G3, G6, K3, K6
6th	H4, H5, J4, J5
7th	H1, H8, J1, J8
8th	I3, I6, G2, K7
9th	H2, H7, J2, J7
10th	I2, I7, K2, G7

STRESSING SEQUENCE FOR LONG RODS ⚠

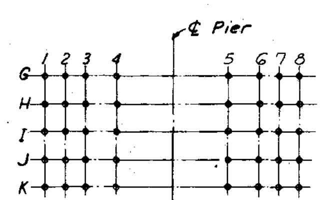
(INTERMEDIATE PIER)

Stage	Tendons
1st	I5
2nd	G5, K5
3rd	H4, J4
4th	H5, J5
5th	G3, K3
6th	G4, K4
7th	I2, I4
8th	H3, J3
9th	G2, K2
10th	H1, J1
11th	G1, K1
12th	I1, I3
13th	H2, J2

STRESSING SEQUENCE FOR LONG RODS

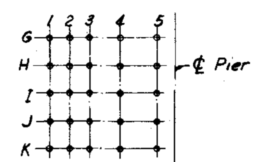
(END PIERS)

Note:
1. The anchor plate shall be of a design and material similar to those furnished by Rod's Inc. or Stressteel Corp.



ROD PATTERN FOR LONG RODS ⚠

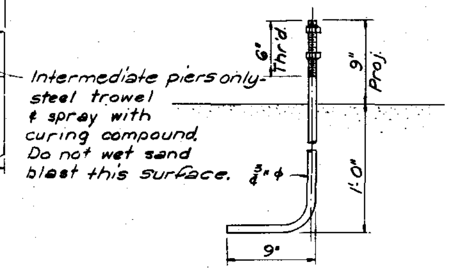
(INTERMEDIATE PIER)



ROD PATTERN FOR LONG RODS

(LEFT END PIER)

(RIGHT END PIER OPP. HAND)

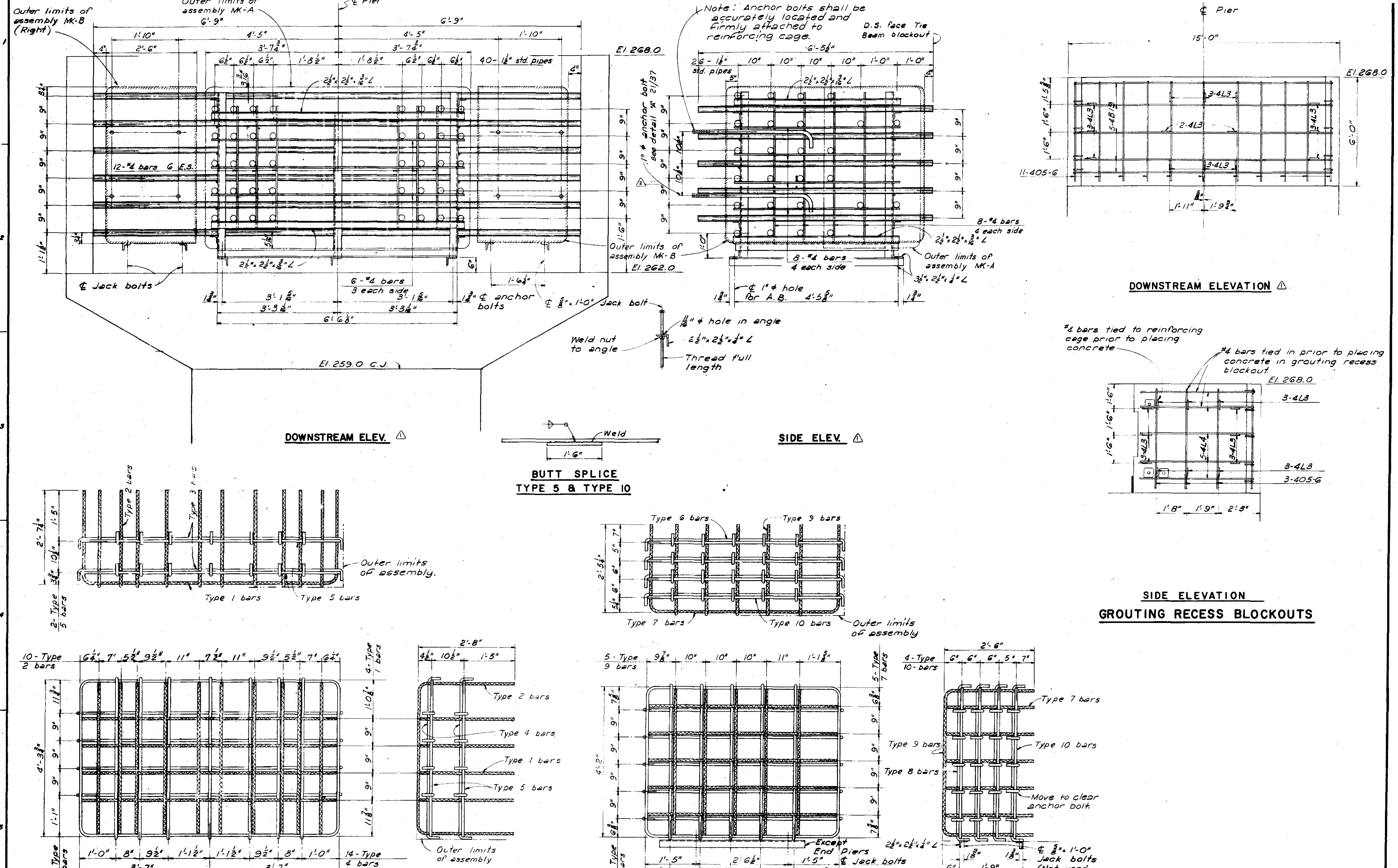


ANCHOR BOLT

TRUNNION ANCHORAGE-PIERS
PLANS, ELEVATION & DETAILS

DESIGNED BY: W. P. JOHNSON, JR.

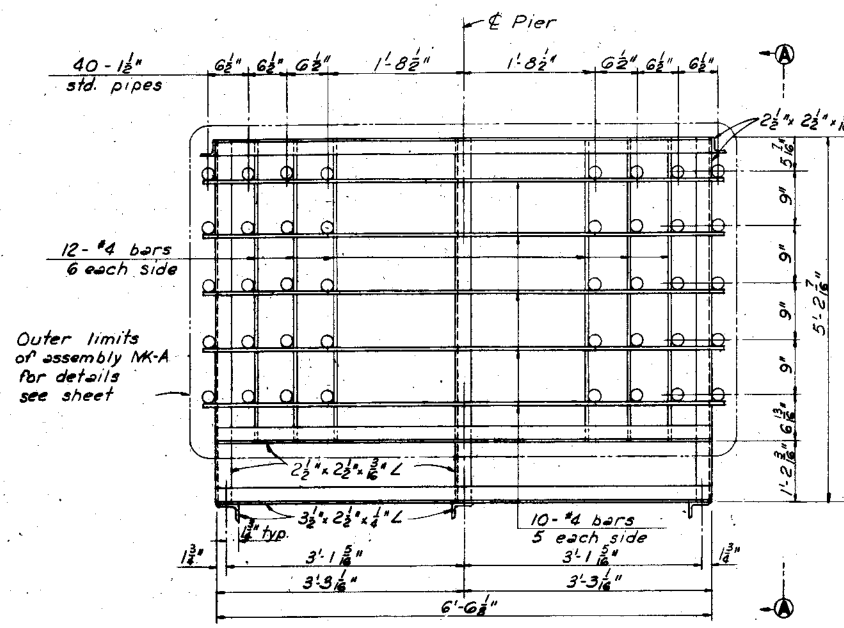
PLATE I



TRUNNION ANCHORAGE-INTERMEDIATE PIER
ASSEMBLIES & ELEVATIONS

DESIGNED BY: W. P. JOHNSON, JR.

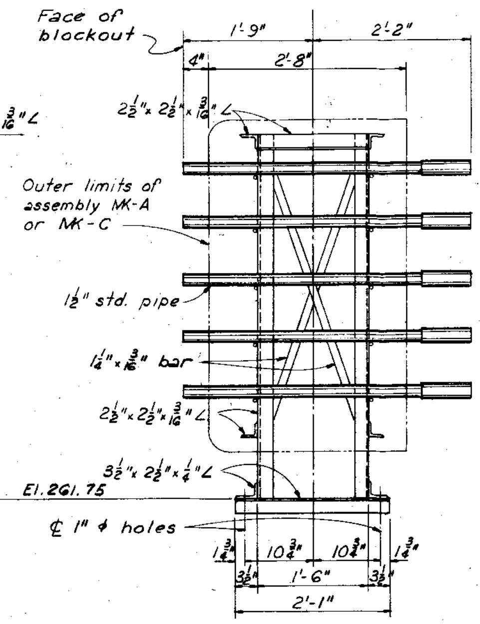
PLATE 2



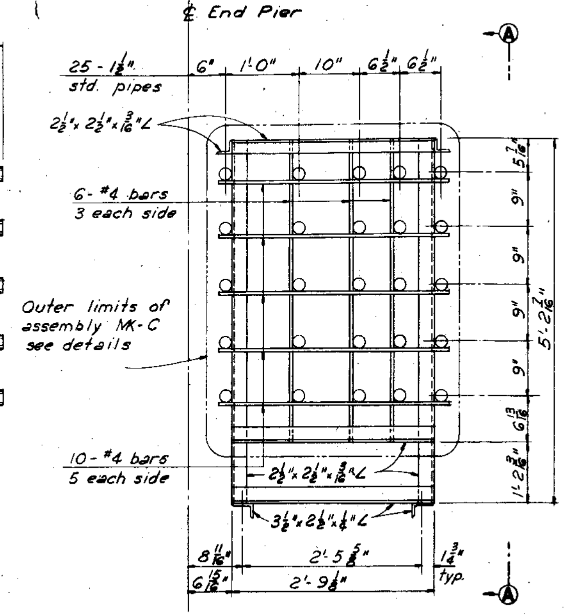
UPSTREAM ELEVATION
INTERMEDIATE PIER

SUPPORT MK-S4
SUPPORT MK-S5 △
(EXCEPT AS NOTED)

Note:
Omit 1½" std. pipes and reinforcing cage (Assembly MK-A) from Support MK-S5



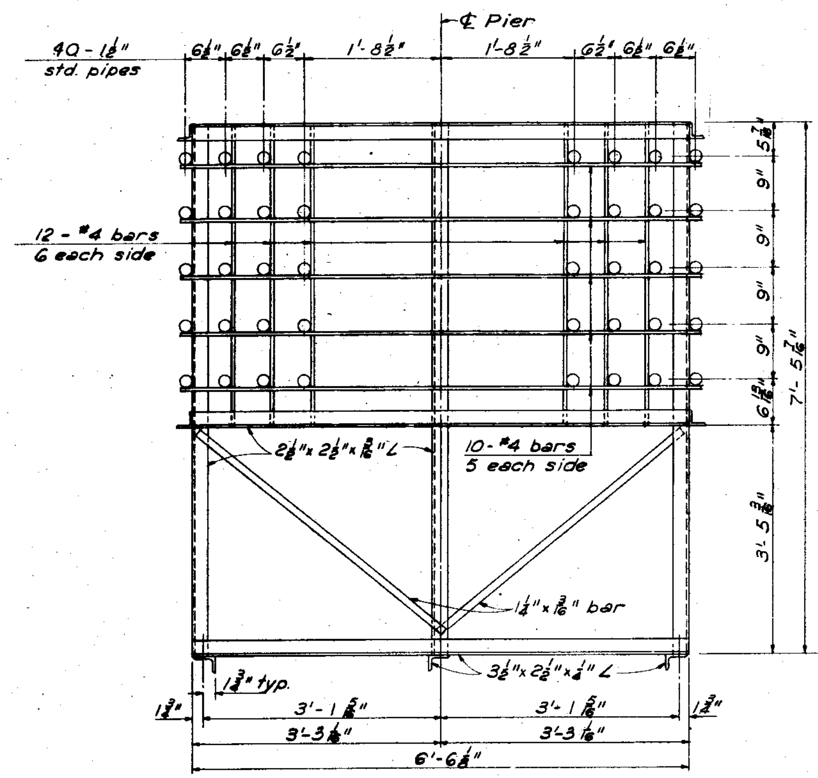
VIEW A-A

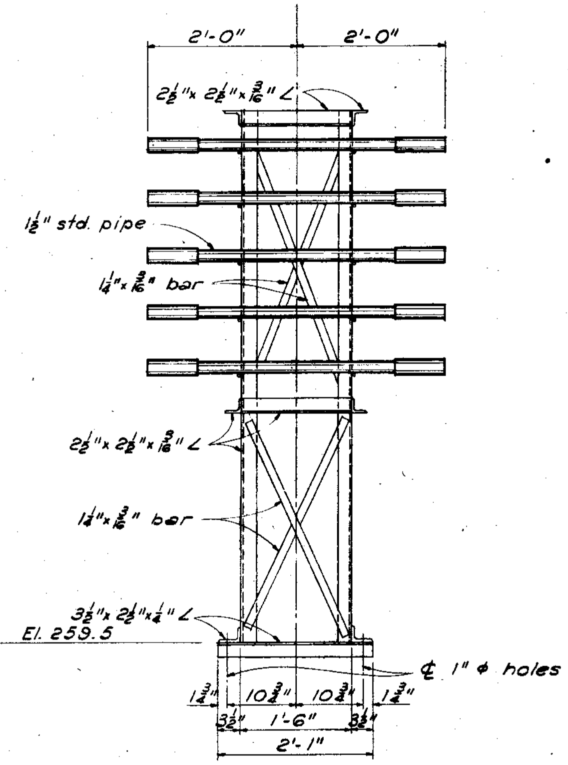


UPSTREAM ELEVATION
LEFT END PIER

SUPPORT MK-S1
SUPPORT MK-S2
(OPP. HAND)
SUPPORT MK-S3
(EXCEPT AS NOTED)

Note:
Omit 1½" std. pipes and reinforcing cage (Assembly MK-C) from Support MK-S3.

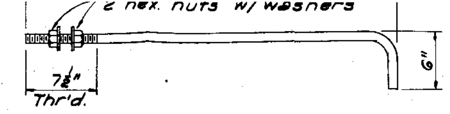




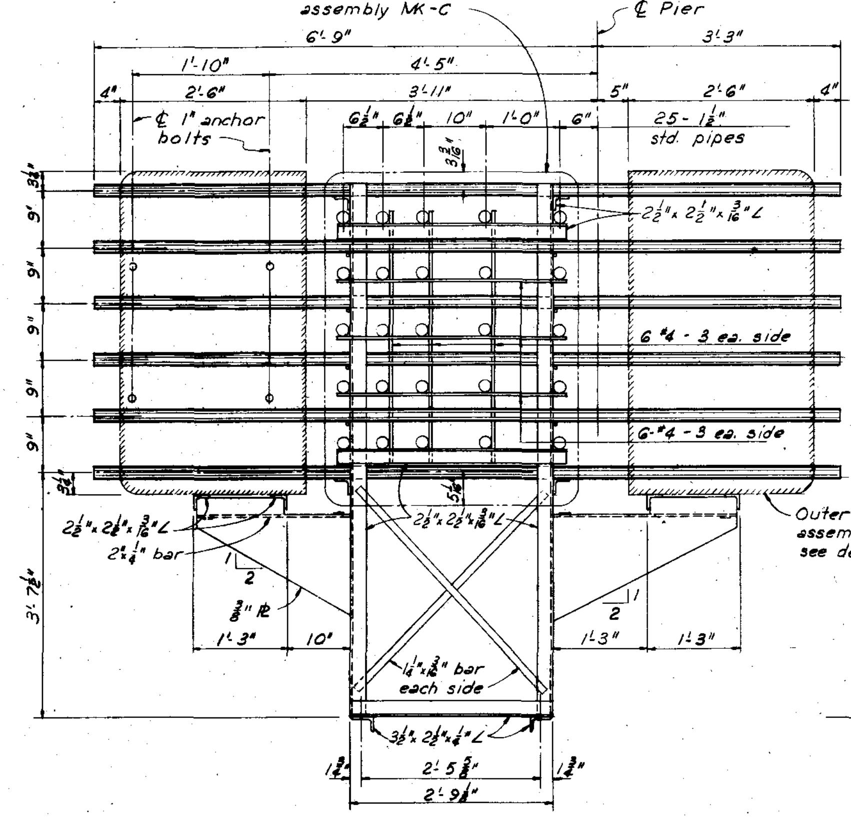
SUPPORT MK-S6 △

TRUNNION ANCHORAGE – PIERS
ELEVATIONS & DETAILS

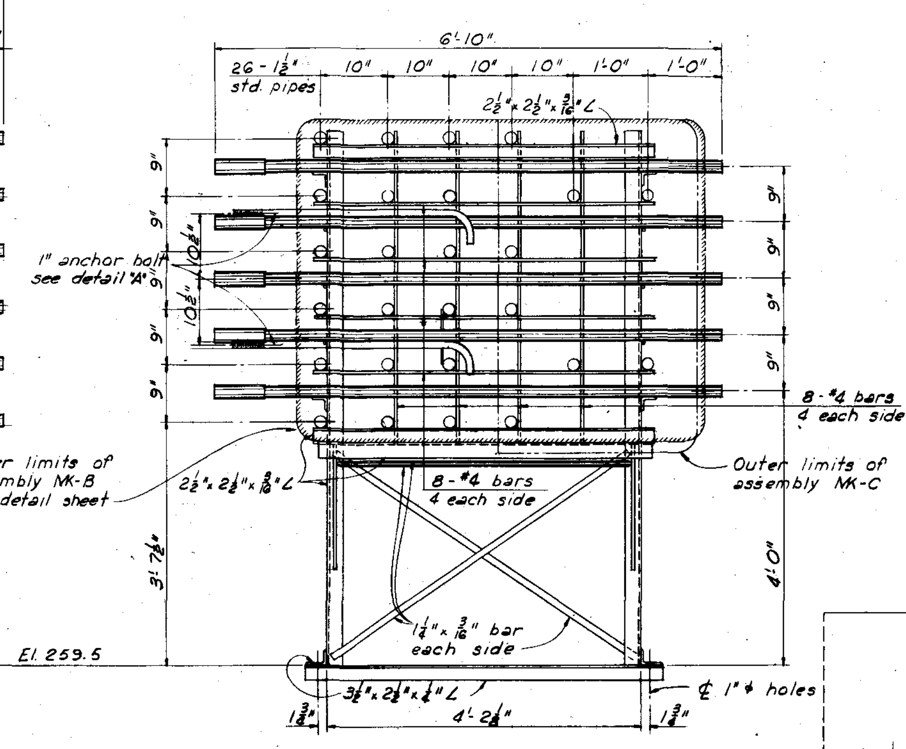
PLATE 3



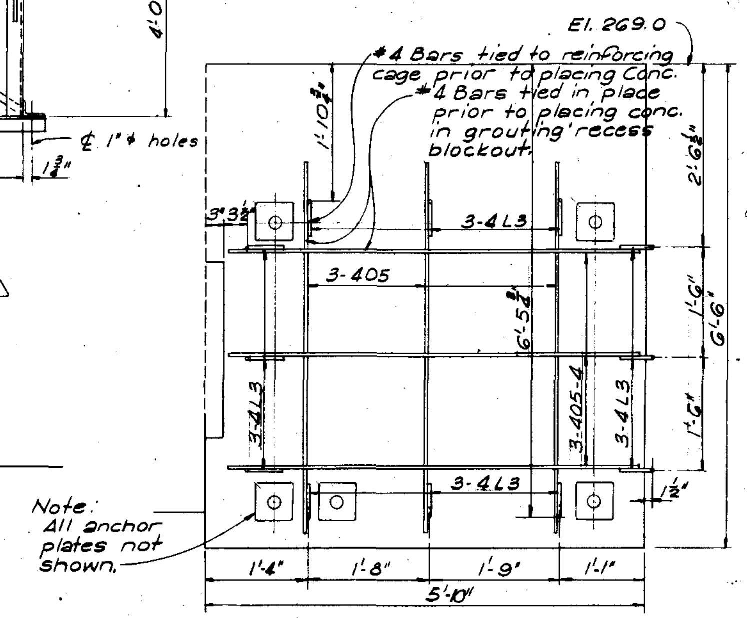
DETAIL "A" △
NO SCALE



DOWNSTREAM ELEVATION △
(LEFT END PIER)
(RIGHT END PIER OPP. HAND)

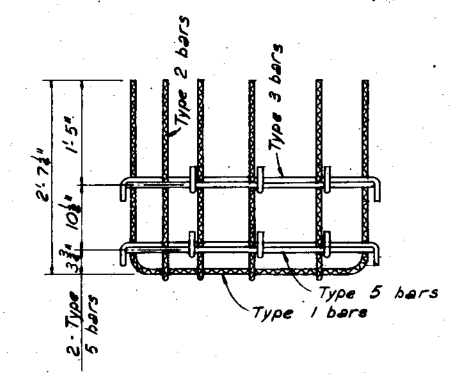


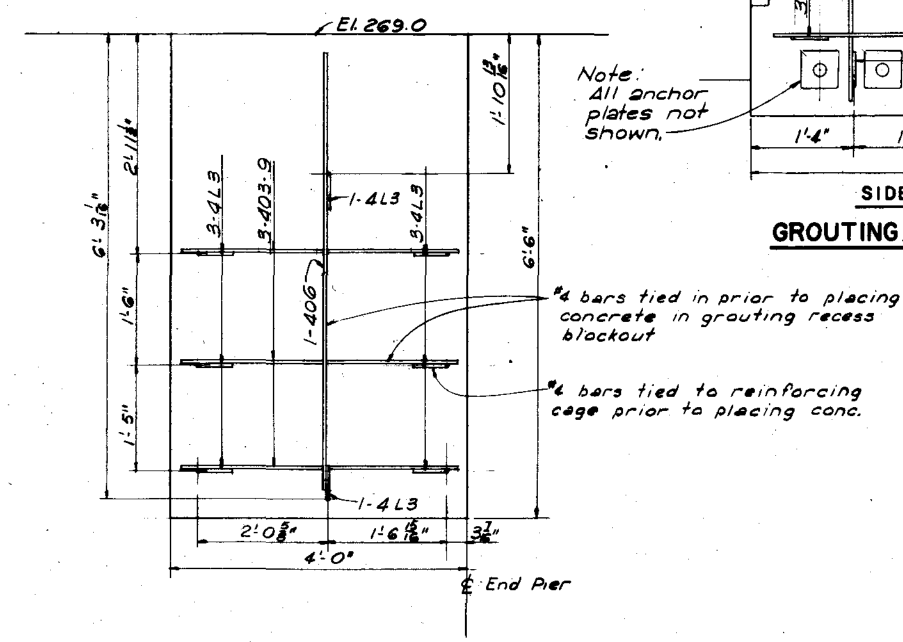
SIDE ELEVATION △



SIDE ELEVATION

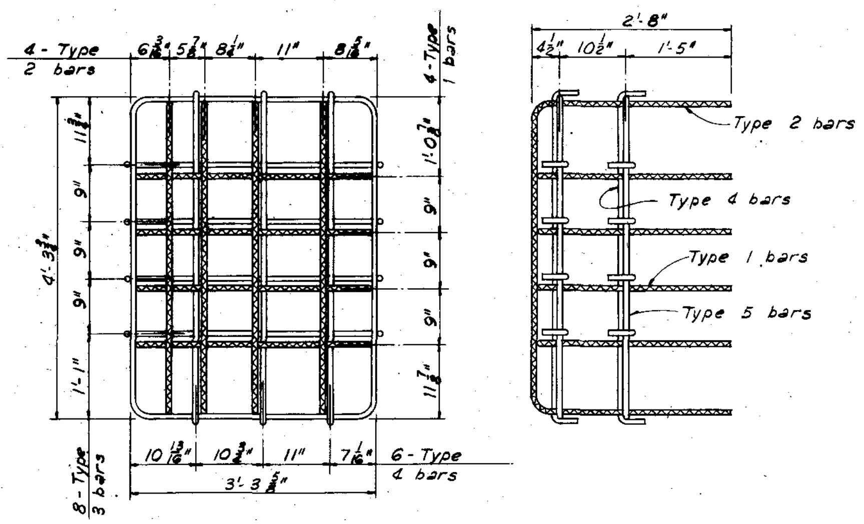
GROUTING RECESS BLOCKOUTS





DOWNSTREAM ELEVATION

GROUTING RECESS BLOCKOUT



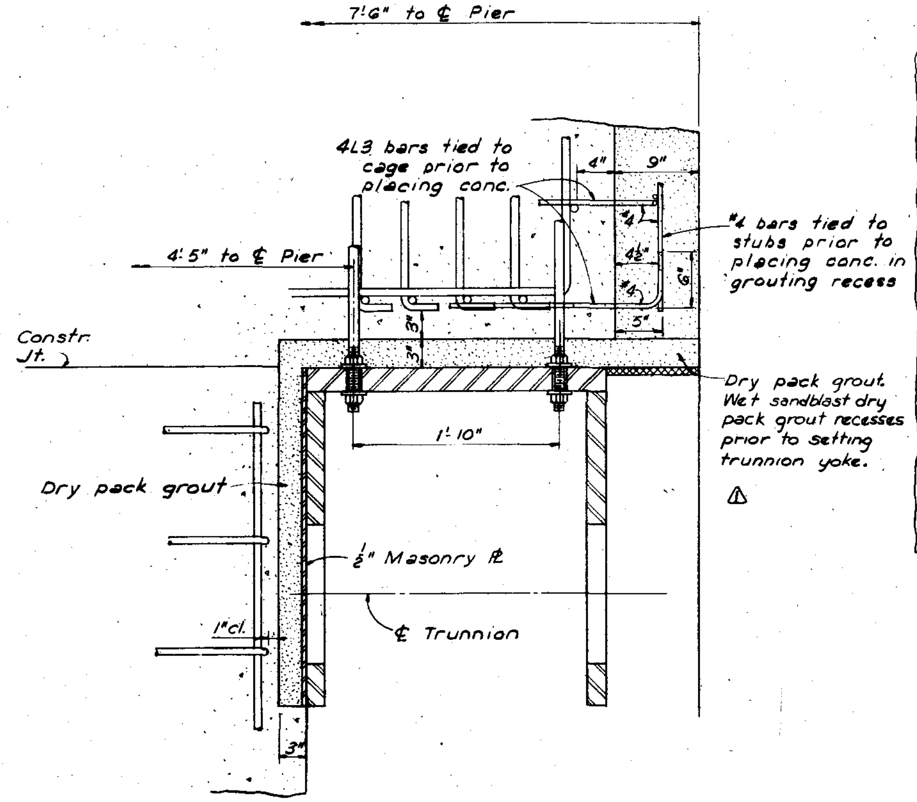
ASSEMBLY MK-C

NOTES:
1. Caution: The spacing, alignment and location of the pipes, frames, and reinforcing steel are critical items of this design, therefore extreme care shall be exercised to comply with all specifications, dimensions, details and notes.
2. Reinforcing steel cages shall be assembled on, and permanently attached to, the pipe support frames in the shop.
3. Care shall be taken that reinforcing cages are assembled to the pipe support frames with the correct orientation.
4. All welding shall be in accordance with the requirements of AWS Standard Specifications for Welded Highway and Railroad Bridges.
5. Reinforcing bars shall be welded at all laying surfaces rather than being tied. Caution - certain bars will have to be assembled on the pipe support frame prior to permanent attachment to the remainder of the reinforcing cage.
6. △ The three reinforcing cages in the intermediate pier thrust beams shall be firmly attached in the field to each other by short lengths of reinforcing rod NOT shown on the drawings.
7. Only those reinforcing bars which bear mark numbers are shown on the bar list. Remainder of reinforcing bars are a portion of the pipe support assemblies.

TRUNNION ANCHORAGE-END PIERS
ELEVATIONS & DETAILS

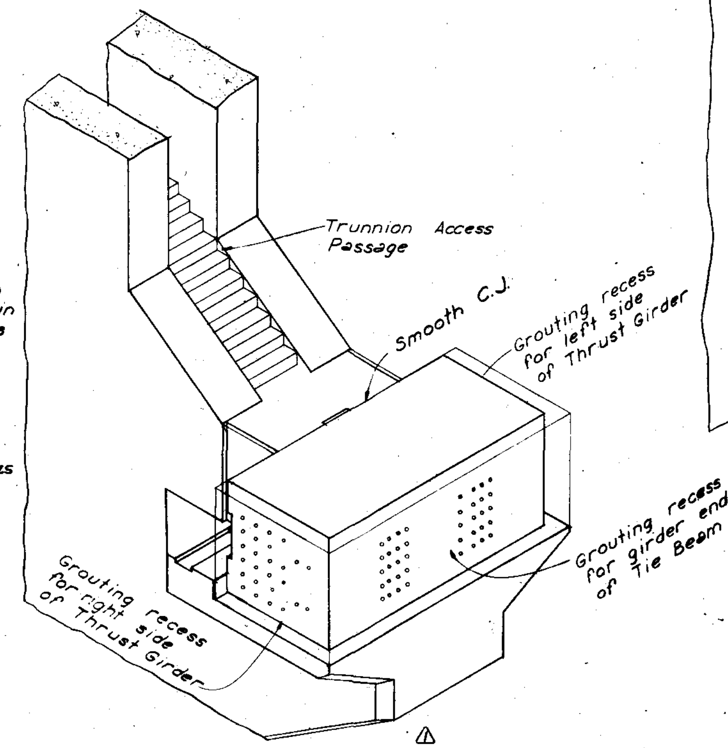
DESIGNED BY: W. P. JOHNSON, JR.

PLATE 4

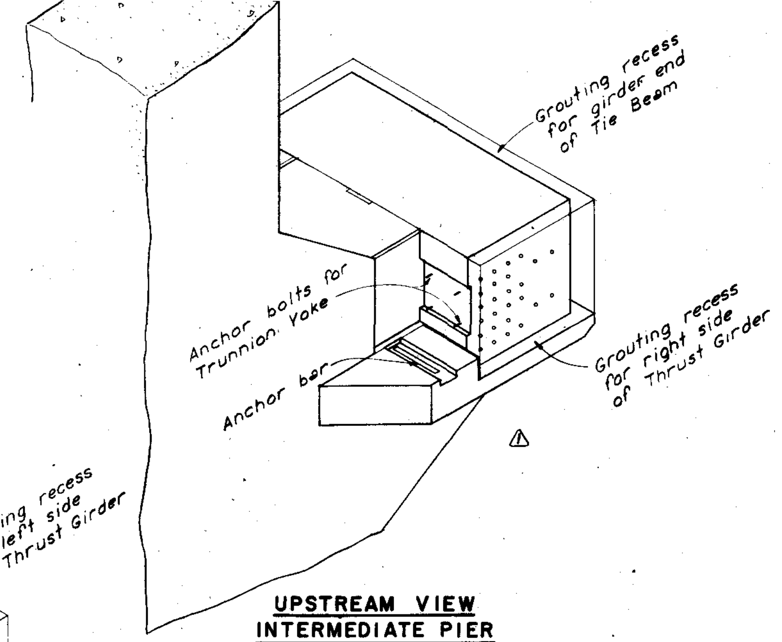


SECTION B-B

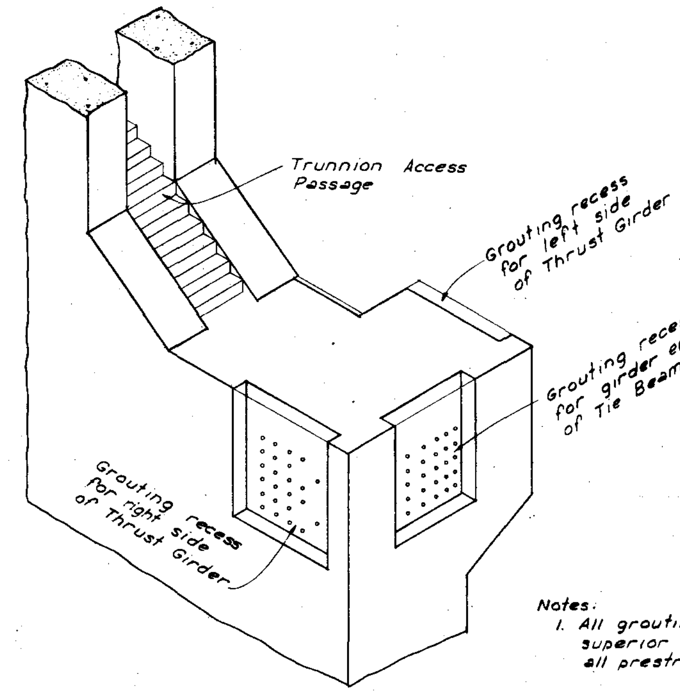
SCALE: 1½ IN. = 1 FOOT



DOWNSTREAM VIEW INTERMEDIATE PIER

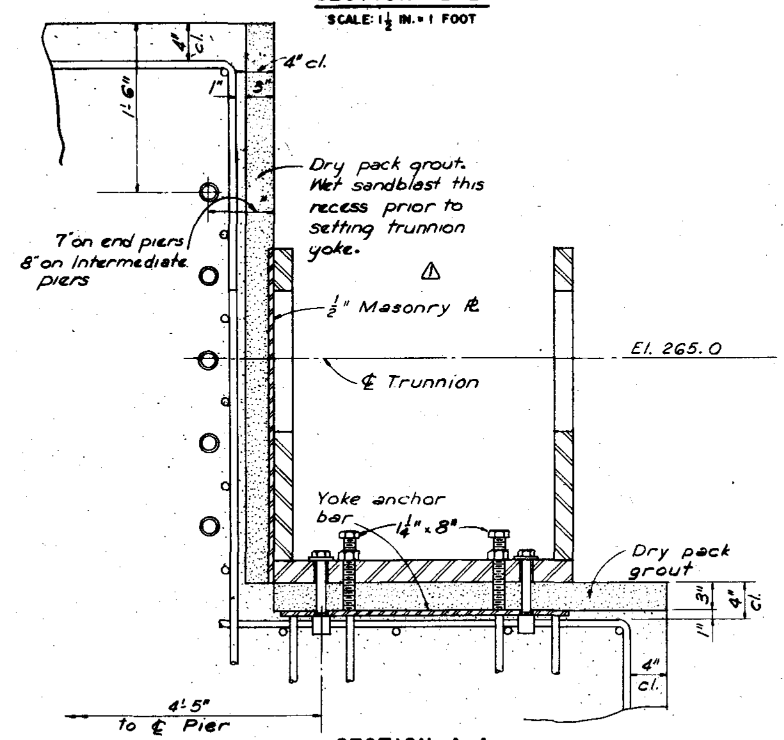


UPSTREAM VIEW INTERMEDIATE PIER



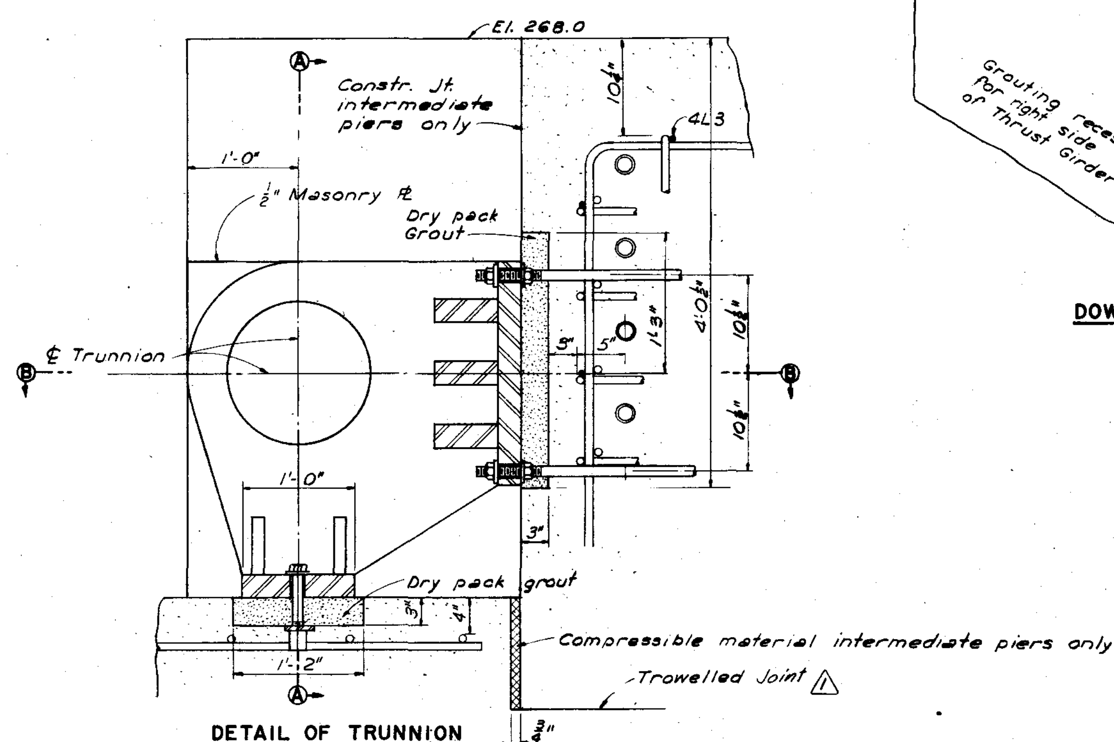
DOWNSTREAM VIEW END PIER

Notes:
1. All grouting recess blockouts to be filled with superior concrete (¾" max. size aggregate) after all prestressing & rod grouting completed.



SECTION A-A

SCALE: 1½ IN. = 1 FOOT



DETAIL OF TRUNNION YOKE GROUT POCKETS

SCALE: 1½ IN. = 1 FOOT

TRUNNION ANCHORAGE-PIERS
CONSTRUCTION PROCEDURE I

DESIGNED BY: W.P. JOHNSON JR.

PLATE 5

CONSTRUCTION PROCEDURE

I. ERECTION

A. GENERAL

1. In the following instructions, reference directions are as follows:

a. **Thrust Girder** - The portion of the prestress rods running normal to the plane of the centerline of the pier at the downstream end of the anchorage.

b. **Tie Beam** - The portion of the prestress rods running parallel to the centerline of the pier.

c. **Anchor End** - The upstream end of the "Tie Beam".

d. **Girder End** - The downstream end of the "Tie Beam".

e. **Right and Left** - The right and left side of the pier as one faces downstream.

2. Each component will be inspected thoroughly, immediately prior to installation on the pier, to ascertain that all individual parts are as detailed on the drawing and that the component has not been damaged in handling, shipment or storage. Any component not passing this inspection will be immediately removed from the job or defaced in such a manner as to preclude it being used in the anchorage.

3. Each component will be thoroughly steam cleaned, immediately prior to actual installation on the pier, to remove all rust inhibitor from all surfaces.

4. The formed surfaces at the ends of all pipes shall be true and normal to the pipes. They shall be smooth comparable to a steel form. The contractor will be required to grind these surfaces should they not meet these requirements.

B. INSTALLATION INTERMEDIATE PIERS

1. Pier concrete shall be poured to the construction joint at elevations 261.25 and 259.00. Anchor bolts for pipe support assemblies shall be cast in place. They shall be supported in the forms so as to be plumb and accurately located.

2. After each pipe support assembly has been steam cleaned, lift into place on the anchor bolts. Use a lifting beam so as not to distort the assembly.

3. Adjust the pipe support assemblies to approximate grade and alignment.

4. Steam clean each pipe and install in the intermediate pipe support frames by inserting into the coupling sleeves of each pipe support assembly. Any splices made in these pipes shall be reamed as shown on the drawings.

5. Adjust all frames to exact grade and alignment.

6. Insure that the frames meet the following requirements;

a. Vertical centerline of tie beam rod pattern coincides with the plane of the centerline of pier from thrust girder end to anchor end.

b. Tie beam pipes are truly horizontal.

7. After all adjustments are completed, insure that the beam pipes are positioned in couplings as shown on the drawings. Tackweld all pipes at couplings and intermediate pipe support frames.

8. Clean out each pipe and place a temporary plug in each end of each pipe. Use a plug which may be easily removed at a later stage.

9. Place pier reinforcing steel, form and place concrete to elevations 262.0 and 270.0. Insure that the forms at ends of pipes are in intimate contact with pipe ends so that pipes will not be blocked by mortar and a smooth bearing surface will be formed. Anchor bolts at elevation 262.0 for the thrust girder pipe support assembly shall be cast in place. They shall be supported in the forms so as to be plumb and accurately located.

10. The horizontal surface at elevation 262.0 shall be steel trowelled. Immediately prior to the installation of the pipe support assembly this surface shall be given a generous spraying with curing compound. This surface shall not be sand blasted or roughened by any operation.

11. After the concrete of the tie beam has cured not less than six (6) days, the thrust girder pipe support assembly may be installed. Remove the temporary plugs in the tie beam pipes. Steam clean and lift the thrust girder pipe support assembly into place using a lifting beam.

12. Adjust the pipe support assembly to approximate grade and alignment.

13. Steam clean and install the stub pipes forming the extension of the tie beam.

14. Adjust the frame to exact grade and alignment.

15. Insure that the assembly meets the following requirements:

a. Thrust girder pipes are truly normal to the plane of the centerline of the pier.

b. Centerline of thrust girder rod pattern is plumb.

c. Tie beam pipe extensions are aligned and horizontal.

16. After all adjustments are completed, insure that the tie beam pipe extensions are positioned in the couplings as shown on the drawings. Tackweld all pipes at couplings and pipe support frame.

17. Clean out each pipe and place a temporary plug in each end of each pipe.

18. Place the thrust girder reinforcing cages and position using the jack bolts. When these cages are in position weld firmly to the pipe support frame and reinforcing cage using short pieces of reinforcing bar.

19. Insure that the forms at the ends of pipes are in intimate contact with pipe ends so that the pipes will not be blocked by mortar and a smooth bearing surface will be formed for the anchor plates.

20. The contractor will not stress the anchorage before he places concrete to elevation 279.25. The pier nose will not be placed until after the rods are stressed and grouted.

C. INSTALLATION END PIERS

1. Pier concrete shall be poured to the construction joint at elevations 261.25 and 259.00. Anchor bolts for pipe support assemblies shall be cast in place. They shall be supported in the forms so as to be plumb and accurately located.

2. After each pipe support assembly has been steam cleaned, lift into place on the anchor bolts. Use a lifting beam so as not to distort the assembly.

3. Adjust the pipe support assemblies to approximate grade and alignment.

4. Steam clean each pipe and install them in the intermediate pipe support frames by inserting into the coupling sleeves of each pipe support assembly. Any splices made in these pipes shall be reamed as shown on the drawings.

5. Adjust all frames to exact grade and alignment.

6. Insure that the frames meet following requirements:

a. Vertical centerline of tie beam rod pattern is parallel with the plane of the centerline of pier from thrust girder end to anchor end.

b. Tie beam pipes are truly horizontal.

c. Thrust girder pipes are truly normal to the plane of the centerline of the pier.

d. Centerline of thrust girder rod pattern is plumb.

7. After all adjustments are completed, insure that the beam pipes are positioned in couplings as shown on the drawings. Tackweld all pipes at couplings and intermediate pipe support frames.

8. Clean out each pipe and place a temporary plug in each end of each pipe. Use a plug which may be easily removed at a later stage.

9. Place pier reinforcing steel, form and place concrete to elevation 270.0.

10. Insure that the forms at the ends of pipes are in intimate contact with pipe ends so that the pipes will not be blocked by mortar and a smooth bearing surface will be formed for the anchor plates.

11. The contractor will not stress the anchorage before he places concrete to elevation 279.25. The pier nose will not be placed until after the rods are stressed and grouted.

II. STRESSING OPERATION

A. GENERAL

1. The stressing operation may begin when all of the following conditions prevail:

a. All superior quality concrete surrounding the anchorage has been placed.

b. The superior quality concrete encasing the stress rod chases has a compressive strength of not less than 4000 psi.

c. The superior quality concrete encasing the stress rods is not less than 10 days old.

d. Pier construction, except the pier nose, has been completed to elevation 279.25.

2. The stressing of all the rods in the anchorage shall be done from the girder end of the anchorage. The contractor has the option of stressing the thrust girder from the right or left side, however, all rods in the girder on any pier will be stressed from the same side.

3. The thrust girder shall be stressed first.

4. The order of stressing the individual stress rods shall be that shown on the drawings under the detail titled "Stressing Sequence". There shall be no variation from this sequence.

5. Stress shall be applied simultaneously and in equal amounts to each rod listed in the "Stressing Sequence" as comprising one stage.

B. PROCEDURE

1. Remove the temporary plugs from the ends of all pipes.

2. Ream the ends of all pipes with a burring reamer until the end of the pipe is 1/16" to 1/8" below the surface of the concrete.

3. Insure that the surface of the concrete around each pipe end is a smooth plane surface at right angle to the axis of the pipe. If necessary the surface shall be ground to insure this condition.

4. Insure that the interior of the pipe is free of debris and clean.

5. Clean the stress rods to remove the rust inhibitor from the entire surface of the rod.

6. Clean the wedge nut to remove rust inhibitor from all surfaces.

7. Insert stress rods into all chases formed by the pipes. Take care during this portion of the operation that the requirements of the specifications, relative to the care of the rods, are complied with.

8. Place anchor plate and wedge nut on each end of each stress rod and bring up snug to concrete. Keep to a minimum the amount of rod projecting from the anchor end.

9. Orient the wedge nut on the anchor end of each rod in such a manner that a three foot length of riser pipe can be attached to each nut during the grouting operation.

10. Assemble wedge pin, jack chair, and jack on stress rods.

11. Apply an initial load of 10,100 pound (10% of final jack load), as indicated by pressure gauge.

12. Wait one minute. During this period check orientation and seating of anchor plates and nuts, and take initial elongation readings.

13. After one minute wait, proceed to tension stress rod to final load of 101,000 pounds per stress rod. This load shall be applied at a uniform rate and over a period of not less than three (3) minutes. The applied force will be measured by both the elongation of the stress rod and the hydraulic pressure applied. Reasonable agreement between these two means of measurement shall be obtained. (Final elongation shall be at least 90% of the total required, based on information relative to physical characteristics of the rods required by the specifications.)

14. The final load of 101,000 pounds per stress rod will be held by the jack for a period of one minute before proceeding with any further operation.

15. After waiting one minute, recheck the stress in the rod. If there has been a loss the rod will again be brought up to the prescribed load and held for one minute.

16. If there has been no loss in load proceed to set the wedge nut by driving the wedge pin.

17. Slowly release the load on the jack.

18. If there is any sign of slippage of the stress rod, bring the jack back up to load and redrive the wedge pin to more securely set the wedge nut.

19. Retract jack ram, remove jack assembly from rod and proceed to next series of rods in "Stressing Sequence".

20. After stressing all rods in thrust girder proceed to stress rods in tie beam.

21. After completion of all stressing on pier, place a temporary protective covering over rod end and proceed to next pier.

22. At least two weeks after the completion of the stressing of all the rods in an anchorage, the rods will again be loaded to determine if any loss in stress has occurred in any of the rods.

23. The order of loading the rods shall be the same as used in the initial tensioning.

24. The load shall be measured by the hydraulic pressure applied.

25. If any loss in stress has occurred the rod shall be brought up to the initial tensioning load and the additional elongation shall be taken up by rotation of the wedge nut.

26. If any rod has lost more than 5% of the final jack load the contractor may, at the Contracting Officer's discretion, be required to wait another two weeks and again check the rods in the pier for loss in load. If loss of tension occurs twice in the same rod, the contractor will replace the stress rod and fittings affected and deface the parts replaced in such a manner as to prevent their re-use.

27. If all rods check, to the satisfaction of the Contracting Officer, the Contractor may proceed to the grouting operation.

28. Under no conditions shall the stress rods at any time be flame cut, used as an electrical ground, welded to, or cold chiseled.

29. After the grouting operations, the stress bars will be cut off with a minimum of one inch projecting beyond the nut. This cut shall be made using an abrasive cutoff wheel.

III. GROUTING OPERATION

A. GENERAL

1. Grouting of the stress rod chases shall be accomplished from the same end of the rod as was used for stressing operation.

2. The contractor shall take precautions to insure that a minimum amount of grout is lost when disconnecting the grouting equipment from each stress rod chase.

B. GROUTING

1. Install a pipe on the wedge nut at the anchor end of each rod so as to form a riser pipe extending a minimum of 2'-0 and a maximum of 3'-0 above the rod chase.

2. Attach a high pressure hose connection to the wedge nut at the stressing end of each rod.

3. Pump grout into chase through high pressure hose connection until grout of the normal consistency is observed coming out of riser pipe installed on the anchor end.

4. Grouting shall be done by starting at the bottom row of each rod pattern and progressing upward.

IV. STRESS ROD PROTECTIVE COVERING

A. GENERAL

1. After the grouting procedure has been completed, the stress rod protective covering shall be placed to a surface flush with the pier.

2. The contractor shall be certain that the surfaces or blockouts are clean and free of any dirt, debris or excess grout. Any damage to the trowelled surface of the intermediate piers shall be repaired prior to placing of concrete.

B. PROCEDURE THRUST GIRDER

1. Number 4 (four) steel reinforcing bars shall be tied to the stubs provided in order to form a mat of bars approximately 18 inches center to center each way and 4 inches clear of the finished outside surface.

2. Superior quality concrete of 3/4" maximum aggregate size shall be placed in the end pier blockouts and at each end and the downstream surface of the intermediate pier thrust girder.

3. The contractor shall take precautions to insure that the concrete is of maximum density, and has a smooth, flush and matching exterior surface.

C. PROCEDURE PIER NOSE

1. The pier nose shall be constructed in the normal manner after the prestressing and grouting operations have been completed.

2. The pier may be constructed to elevation 288.75 prior to the stressing or grouting operations by electing to use the optional pier nose construction joint from elevation 279.25 to elevation 288.75.

3. No pier concrete may be placed above elevation 288.75 until the pier nose has been completed to this elevation.

V. ADJUSTING AND GROUTING TRUNNION YOKES

A. GENERAL

1. The trunnion yokes shall be set in the position shown.

2. The gate shall be completely installed.

B. PROCEDURE

1. The gate shall be operated and any adjustments of the trunnion yoke necessary to provide for a freely working gate shall be made using the adjusting bolts and nuts.

2. After all adjustments have been made, the recesses around the trunnion yoke shall be filled with dry pack grout of minimum strength of 3,000 psi. All exposed surfaces of the dry pack grout shall be finished to a smooth, level and flush surface.

TRUNNION ANCHORAGE-PIERS
CONSTRUCTION PROCEDURE II

VITA

William Preston Johnson, Jr.

Candidate for the Degree of

Master of Science

Thesis: DESIGN OF TAINTER GATE PRESTRESSED CONCRETE TRUNNION ANCHORAGES

Major Field: Civil Engineering

Biographical:

Personal Data: Born in Tulsa, Oklahoma, February 25, 1935, the son of William Preston and Selma Jo Johnson.

Education: Attended Tulsa public school system, graduating from Central High School in May, 1953. Attended Rice University, Houston, Texas, September, 1953 to May, 1958. Received Bachelor of Arts degree in May, 1957 and Bachelor of Science in Civil Engineering in May, 1958. Attended Oklahoma State University June, 1961 to May, 1962. Received Master of Science degree in Civil Engineering in May, 1964.

Professional Experience: Employed from June through August, 1956 at Douglas Aircraft Company in Tulsa as a mathematician in the stress analysis group. From June to September, 1957 and June to November, 1958, employed by U. S. Corps of Engineers, Tulsa, as a civil engineer in a contract plans section. From January, 1959 to October, 1960, served as a Lieutenant with the Corps of Engineers, 70th Engineer Company, France, supervising military construction. From November, 1960 to June, 1961 and June, 1962 to the present, employed as a structural engineer, Hydraulic Structures, Tulsa District, Corps of Engineers.